Concrete

SECOND EDITION

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Preface

Portland cement concrete is foremost among the construction materials used in civil engineering projects around the world. [redacted] Concrete is unique among major construction materials in that it is generally designed specifically for a particular project using locally available materials. Therefore, the project engineer has full control and responsibility over the final material used in construction. [redacted]. It is thus essential that engineers acquire a thorough understanding of [redacted] properties of concretes and the procedures that are essential to providing a material of the required quality and durability.

In the past, concrete technology has been taught largely as an empirical science. [redacted] it is the [redacted] text to present a unified view of concrete behavior in light of [redacted]ples, rather than as a series of more or less unrelated facts. For instance, the material on the workability of concrete is prefaced by a general discussion of the principles of rheology; mechanical properties are discussed from the point of view of concrete as a composite (or multiphase) material; and the underlying chemistry of hydration and microstructure of the hardened cement paste are emphasized.

This book is designed primarily for use at the undergraduate level, but it should also serve as a guide for the professional engineer who did not take a formal course on concrete in college. The text is based on the authors' considerable experience in teaching the principles of concrete at the undergraduate level. It includes the most up-to-date information available on new concrete materials, and considerable attention is given to the role of specifications for concrete and concrete materials and the use of test methods for determining concrete properties.

To provide a comprehensive treatment, more material is included than can be covered in detail in one semester. Therefore, the instructor will have to choose to omit certain topics. Chapter 4 contains more chemistry than might be considered desirable for an undergraduate course and could be treated in less detail by concentrating only on the reactions of the principal cement compounds. Also, those sections in Chapter 16 dealing with mechanisms of shrinkage, strain-rate dependence, and creep could be

omitted if desired, although they contribute to a more basic understanding of the material. Chapters 21 and 22 could be passed over without loss of continuity, while parts of Chapters 15, 18 and 20 could also be omitted, depending on the instructor's interests and the emphasis of the curriculum.

The book is divided into three main parts:

1. The properties of the constituent materials: cements, hardened cement paste, aggregates, water and admixtures.
2. Proportioning of concrete mixes and construction practices: mixing, transporting, placing and consolidating, and curing.
3. The properties of hardened concrete: strength and fracture, fatigue, creep and drying shrinkage, and durability.

In view of the fact that SI units are in use in Canada, and are being adopted in the United States, we have chosen to use SI units as the primary system. Conversion to the English system is given throughout the text, as appropriate. Numerical problems have been given using both sets of units.

Throughout the text, reference is made to national standards developed by the American Society for Testing and Materials (ASTM) and the Canadian Standards Association (CSA). Also, considerable use is made of the recommendations and reports of the American Concrete Institute (ACI). ASTM and ACI documents are the principal standards in the United States, but are also widely used elsewhere. In preparing the text, reference was made to the most up-to-date editions of the standards and recommendations available. These are subject to frequent revision and the reader should therefore refer to the most recent editions, which may differ in some details from those referred to in the text.

A selected bibliography is provided with each chapter as a guide to further reading and to provide a point of entry to the original scientific literature. Some exercises are given at the end of each chapter. Selection of these has been difficult since the subject does not lend itself to numerical problems, but they have been chosen to emphasize the important aspects of each chapter.

It has been 22 years since publication of the first edition. Much progress has been made in many areas of concrete technology, necessitating some modification in coverage. For those familiar with the first edition, very little has been removed. The text now contains new, separate chapters on mineral admixtures, cement–polymer composites, and fiber reinforced concrete, subjects that had been included as part of other chapters in the first edition. The chapters on cements, aggregates, chemical admixtures, concrete construction practices, and durability have undergone major revisions and the information on architectural concrete has been moved into Chapter 20 on Concrete for Special Applications. Several chapters that addressed the response of concrete to stress have been consolidated into a single, thoroughly revised, chapter. And, the text now includes an all new chapter on high-strength concrete.

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CHAPTER 1

Concrete as a Material

Concrete is a material that literally forms the basis of our modern society. Scarcely any aspect of our daily lives does not depend directly or indirectly on concrete. We may live, work, study, or play in concrete structures to which we drive over concrete roads and bridges. Our goods may be transported by trucks traveling on concrete superhighways, by trains that run on rails supported on concrete crossties, by ships that moor at concrete piers in harbors protected by concrete breakwaters, or by airplanes landing and taking off on concrete runways. Water for drinking and for raising crops is stored behind massive concrete dams and is distributed by systems of concrete waterways, conduits, and pipes. The water thus stored may also be used to generate electric power. Alternatively, electricity can be generated by burning coal in power stations built from concrete or by harnessing the power of the atom within massive reinforced concrete pressure vessels. Concrete plays a key role in oil production, being used to construct massive off-shore platforms for drilling and production (Figure 1.1).

We take concrete for granted in our everyday activities and tend to be impressed by the more dramatic impacts of technology. However, it can be truly said that many of the achievements of our modern civilization have depended on concrete, just as many of the enduring achievements of the earlier civilization of Rome were made possible by the use of the forerunner of modern concrete. The word *concrete* comes from the Latin verb "concretus," which means to grow together.

1.1 THE NATURE OF CONCRETE

Concrete is a composite material (see Table 1.1) composed of coarse granular material (the aggregate or filler) embedded in a hard matrix of material (the cement or binder) that fills the space between the aggregate particles and glues them together. Aggregates can be obtained from many different kinds of materials, although we mostly make use of the materials of nature—common rocks. They are essentially inert, filler materials that, for convenience, are separated into fine and coarse fractions. Similarly, the cement can be formulated from many diverse chemicals. "Cement" is a generic term that can apply to all binders. Therefore, descriptors must be used to qualify this term when referring to specific materials. A civil engineer may have cause to use

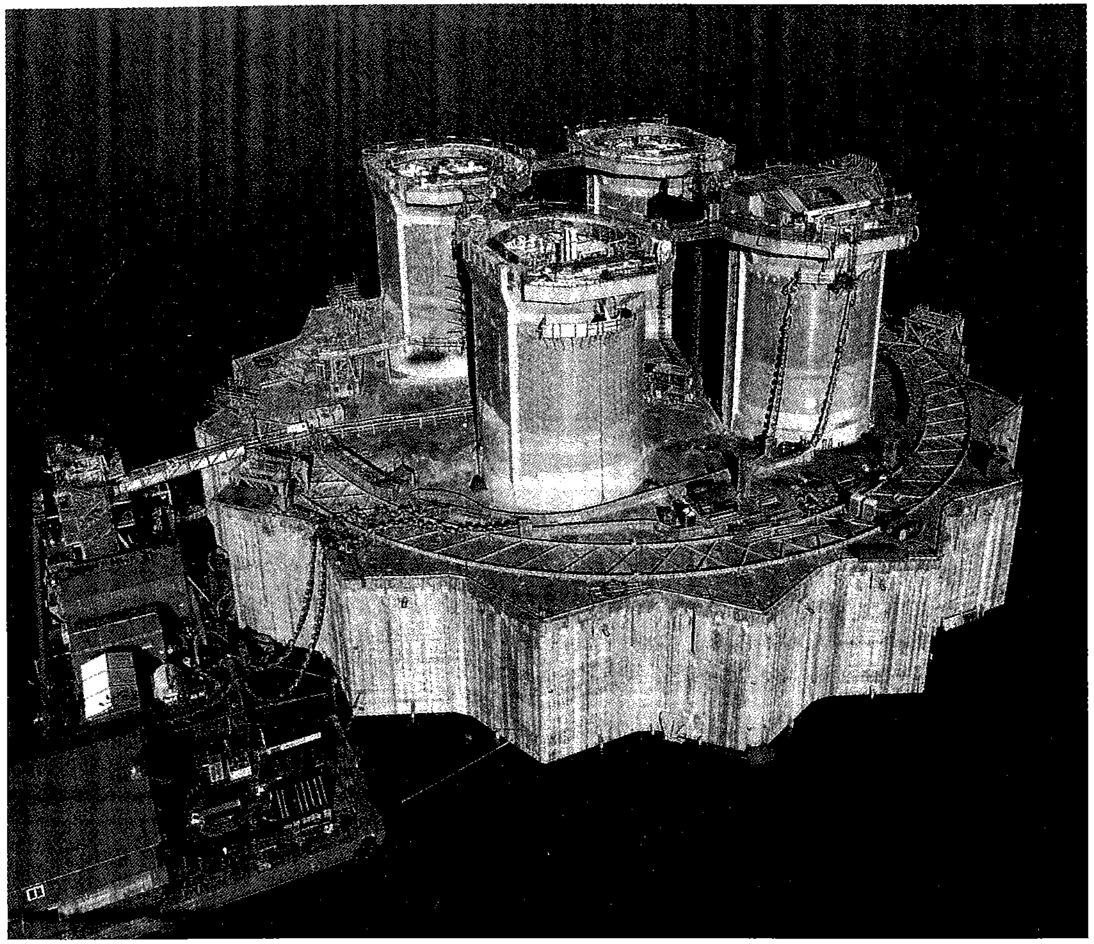


FIGURE 1.1

Gravity base structure for the Hibernia off-shore oil platform. (Photograph courtesy of Hoff Consulting LLC, Clinton, MS.)

TABLE 1.1 Definitions for Concrete

Concrete	=	*Filler*	+	*Binder*
Portland cement concrete	=	Aggregate (fine and coarse)	+	Portland cement paste
Mortar	=	Fine aggregate	+	Paste
Paste	=	Cement	+	Water

portland cement concrete, calcium aluminate cement concrete, or polymer concrete, where the binders are portland cement, calcium aluminate cement, or a polymer resin, respectively. In concrete construction, the engineer will use portland cement concrete 95% of the time. (The name refers to a family of calcium silicate cements whose origins are described in Chapter 2.) Thus, for convenience, we will often drop the name *portland* throughout the text and use a qualifying descriptor only when dealing with other kinds of cement and concrete.

1.2 ADVANTAGES OF CONCRETE

Concrete is the predominant material used in construction (see Table 1.1.). It competes directly with all other major construction materials—timber, steel, asphalt, stone, etc.—because of its versatility in applications. To meet the demand for concrete and concrete products, 105 million metric tons of cement were consumed in the United States in 2000 (Table 1.2) and 8 million metric tons in Canada.

These figures point clearly to inherent advantages attending the use of concrete. The major advantages and disadvantages of concrete are summarized in Table 1.3. Typical properties of concrete are given in Table 1.4 and compared to other construction materials in Table 1.5.

It should be remembered, however, that concrete properties can vary significantly from the figures given, depending on the choice of materials and proportions for a particular application. For example, subsequent chapters will show how concrete can be designed to have compressive strengths <10 MPa (1500 lb/in.^{2}) or >100 MPa (15,000 lb/in.^{2}) with concomitant changes in modulus of elasticity.

The ability of concrete to be cast to any desired shape and configuration is an important characteristic that can offset other shortcomings. Concrete can be cast into soaring arches and columns, complex hyperbolic shells, or into massive, monolithic sections used in dams, piers, and abutments. On-site construction means that local materials can be used to a large extent, thereby keeping costs down. Cement costs only about 7–10 cents/kg (3–4 cents/lb) (2001) and aggregates less than 2 cents/kg (<1 cent/lb). Furthermore, by fabricating concrete on site, its properties may be tailored for the

TABLE 1.2 Quantities of Materials Used in U.S. Construction, 2000

	Volume		*Weight*	
Material	10^6 m^3	10^6 ft^3	10^6 metric tons	10^6 tons
Timber	107	3780	–	–
Concrete	275	9700	640	700
Cement	33	1180	105	116
Steel	2	60	13	15
Brick and clay products	–	–	39	43
Building stone	0.3	12	1	1
Nonferrous metals	–	–	2	2
Asphalt	–	–	29	32

TABLE 1.3 Advantages and Disadvantages of Concrete as a Construction Material

Advantages	*Disadvantages*
Ability to be cast	Low tensile strength
Economical	Low ductility
Durable	Volume instability
Fire resistant	Low strength-to-weight ratio
Energy efficient	
On-site fabrication	
Aesthetic properties	

specific application. On the other hand, on-site production is a mixed blessing because the quality of concrete must be carefully controlled. Environmental conditions fluctuate, so that it is difficult to assure uniform processing of concrete throughout a job. Constituent materials are less carefully characterized than they might be and can have undesirably high variations in properties. The use of an unskilled or semiskilled work force means that in the absence of proper supervision on the job site, undesirable practices may be adopted and tolerated.

Casting of concrete can also be adapted to factory-controlled production. Precast building elements for standardized low-cost building systems are more common in European countries, but have also been developed in the United States. Precast concrete block has become a very popular building element, and precast concrete pipe is widely used in drainage, sewage, and water-supply projects. Precast, prestressed concrete beams, girders, and panels in various configurations are used increasingly in many structures. Precast concrete can be produced more uniformly with closer tolerances compared to concrete cast on site, but requires a more skilled work force and generally more sophisticated equipment.

Good-quality concrete is a very durable material and should remain maintenance free for many years when it has been properly designed for the service conditions and properly placed. Through choice of aggregates, or control of paste chemistry and microstructure, concrete can be made inherently resistant to physical attack, such as

TABLE 1.4 Typical Engineering Properties of Structural Concrete

Compressive strength	= 35 MPa (5000 lb/in.^{2})
Flexural strength	= 6 MPa (800 lb/in.^{2})
Tensile strength	= 3 MPa (400 lb/in.^{2})
Modulus of elasticity	= 28 GPa (4×10^6 lb/in.^{2})
Poisson's ratio	= 0.18
Tensile strain at failure	= 0.001
Coefficient of thermal expansion	= 10×10^{-6}/°C (5.5×10^{-6}/°F)
Ultimate shrinkage strain	= 0.05–0.1%
Density	
Normal weight	= 2300 kg/m^3 (145 lb/ft^3)
Lightweight	= 1800 kg/m^3 (110 lb/ft^3)

TABLE 1.5 Typical Properties of Construction Materials.[a]

Material	*Density* (kg/m^3)	*Tensile Strength* (MPa)	*Elastic Modulus* (GPa)	*Coefficient of Thermal Expansion* (10^{-6}/°C)	*Thermal Conductivity* (W/m·K)	*Energy Requirement* (GJ/m^3)
Aluminum						
Pure	2800	100	70	23	220	360
Alloy	2800	300	70	23	125	360
Steel						
Mild	7800	400	210	12	50	300
High strength	7800	1900	210	12	50	300
Glass	2500	60	65	6	3	50
Wood						
Soft	350	50	5.5	—	0.2–0.6	—
Hard	700	100	10	—	0.2–0.6	—
Plastic (polystyrene)	1000	~50	~3	72	0.1	—
Rock (granite)	2600	~20(~25[b])	~50	7–9	3	—
Concrete	2300	3(35[b])	~25	10	3	3.4

[a]Conversion factors: kg/m^3 × 0.062 = lb/ft^3; MPa × 145 = lb/in.^{2}; GPa × 0.145 = 10^6 lb/in.^{2}; 10^{-6}/°C× 0.556 = 10^{-6}/°F; W/m·K × 0.578 = Btu/ft·h·°F; GJ/m^3 × 26.9×10^3 = Btu/ft^3.
[b]In compression.

from cycles of freezing and thawing or from abrasion, and from chemical attack, such as from dissolved sulfates or acids attacking the paste matrix or from highly alkaline pore solutions attacking certain aggregates. Judicious use of mineral admixtures greatly enhances the durability of concrete. Unlike structural steel, it does not require protective coatings except in very corrosive environments. It is also an excellent material for fire resistance. Although it can be severely damaged by exposure to high temperatures, it can maintain its structural integrity for a considerable period—long after steel buildings would have suffered irreparable damage.

Because of the high impact of energy costs on material costs and the (usually) accompanying production of carbon dioxide (which contributes to global warming), significant attention is given to the energy requirements of construction materials. In this regard, concrete comes out ahead of most other construction materials (see Table 1.5.). In the first place, concrete requires less energy to produce than does steel. This is because steel is made by high-temperature processes (300 GJ/m^3), whereas only a small component of the concrete, the cement, requires pyro-processing (22 GJ/m^3). The major energy costs of concrete are in cement and reinforcing steel, but the energy consumption of an equivalent steel structural element can be considerably greater. Second, energy requirements can be decreased for concrete by incorporating supplementary cementing materials, such as fly ash, silica fume, and blast furnace slag. These byproducts of other industrial processes not only reduce the quantity of cement required to achieve a given strength, but can be used to attain strengths greater than can be achieved using portland cement alone. Higher strengths translate into smaller cross-sections, and, thus, lower quantities of concrete. Third, concrete buildings can be more energy efficient to operate because of the thermal properties of concrete. Concrete conducts heat slowly and is able to store considerable quantities of heat from the environment that can be released during cool periods. It is possible to design buildings

to take advantage of this thermal inertia. Last, but not least, concrete has considerable aesthetic possibilities that can be expressed through the use of color, texture, shape, and so on. All of these advantages combine to make concrete very versatile and adaptable.

1.3 LIMITATIONS OF CONCRETE

However, concrete does have weaknesses that may limit its use in certain applications and must be allowed for when designing structures. Concrete is a brittle material with very low tensile strength. Thus, concrete should generally not be loaded in tension (except for low bending stresses that may be permitted in unreinforced slabs on grade), and reinforcing steel must be used to carry tensile loads; inadvertent tensile loading causes cracking. The low ductility of concrete also means that concrete lacks impact strength and toughness compared to metals. Even in compression, concrete has a relatively low strength-to-weight ratio, and a high load capacity requires comparatively large masses of concrete, although, since concrete is low in cost, this is economically possible.

The volume instability of concrete must also be allowed for in design and construction. It shows volume stability that is more characteristic of timber and quite unlike that of steel, which is a volume-stable material under normal conditions of service. Concrete undergoes considerable irreversible shrinkage due to moisture loss at ambient temperatures and also creeps significantly under an applied load even under conditions of normal service.

Awareness of these problems with concrete enables us to compensate for them, by using suitable designs and by controlling them, in part, through a suitable choice of materials and construction practices. A great deal of research effort has been devoted to ameliorating these problems and now ready-mixed concrete with compressive strengths of 100 MPa (15,000 lb/in.^{2}) can be routinely produced in some areas. Over the last 30 years, new types of concrete have been developed, such as fiber-reinforced concrete, shrinkage-compensated concrete, and latex-modified concrete. Cement-based materials with flexural strengths exceeding 150 MPa (22,000 lb/in.^{2}) or with tensile strains greater than 1% have been produced.

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CHAPTER 2

Historical Development of Cement and Concrete

From very early times, builders have tried to find materials that could be used to cement stones or bricks together. It was clear to them that this mode of construction would provide much more flexibility than the older method of carefully setting stone blocks one above the other without the use of any cementing material between them. Perhaps the oldest cementing material was simply mud, sometimes mixed with straw, to bind dried bricks together, as used in ancient Egypt. Such construction will only work in very dry climates, as the unburnt bricks and clay have no resistance to water. The Babylonians and Assyrians sometimes used naturally occurring bitumens to bind stones or bricks together. In our own time, a variety of organic cements (epoxies) are similarly used to bind structural elements together. However, in this chapter we only follow the development of cements based on compounds of lime (i.e., *calcareous* cements). In particular, we focus on those cements that are capable of hardening under water (i.e., *hydraulic* cements).

2.1 NONHYDRAULIC CEMENTS

The first calcareous materials to be used as cements in mortars were gypsum and lime. The Egyptians used gypsum mortars in the construction of the Pyramid of Cheops (~3000 B.C.); these were prepared by calcining impure gypsum. When mixed with a small quantity of water, the setting of this material is due to the recombination of the calcined gypsum with the water of crystallization that had been driven off during the burning process:

$$2CaSO_4 \cdot 2H_2O \xrightarrow[\sim 130^\circ C]{heat} 2CaSO_4 \cdot \tfrac{1}{2}H_2O + 3H_2O \tag{2.1}$$

This material was used instead of lime because it required only a relatively low burning temperature (~130°C). Gypsum-based mortars are nonhydraulic; that is, hardening will not take place under water, because gypsum is quite soluble.

Lime mortars were used in Egypt only in the Roman period, but they were used much earlier in Crete, Cyprus, Greece, and the Middle East (6000–12,000 B.C.) These materials were prepared by calcining limestone:

$$CaCO_3 \xrightarrow[\sim 1000°C]{heat} CaO + CO_2 \tag{2.2}$$

Lime mortar is sometimes known as *air mortar*, since it hardens in air. The hardening is due essentially to the evaporation of the mixing water. A secondary reaction is then the subsequent carbonation:

$$CaO + H_2O \rightarrow Ca(OH)_2 \xrightarrow{CO_2} CaCO_3 + H_2O \tag{2.3}$$

This helps to solidify the mass, but only on the surface, and if the lime mortar is well compacted, CO_2 is prevented from penetrating beyond a thin surface layer. It should be noted that although the hardened mortar is quite impervious to water, the hardening will not take place *under* water. Thus, ordinary limes are not hydraulic. Nevertheless, some excellent mortars were produced by this method—so hard that at one time it was assumed that some secret, now lost, was involved in the production of the mortar. Marcus Vitruvius[1] describes in some detail the different grades of plaster used to finish walls in Roman buildings. Later study has shown, however, that the excellent performance of these mortars was not due to a secret in the making of the lime, but to thorough mixing and ramming (a lesson that modern engineers would do well to remember). The Colosseum in Rome and the Pont du Gard at Nîmes (Figure 2.1) are tributes to the quality of at least some Roman mortars.

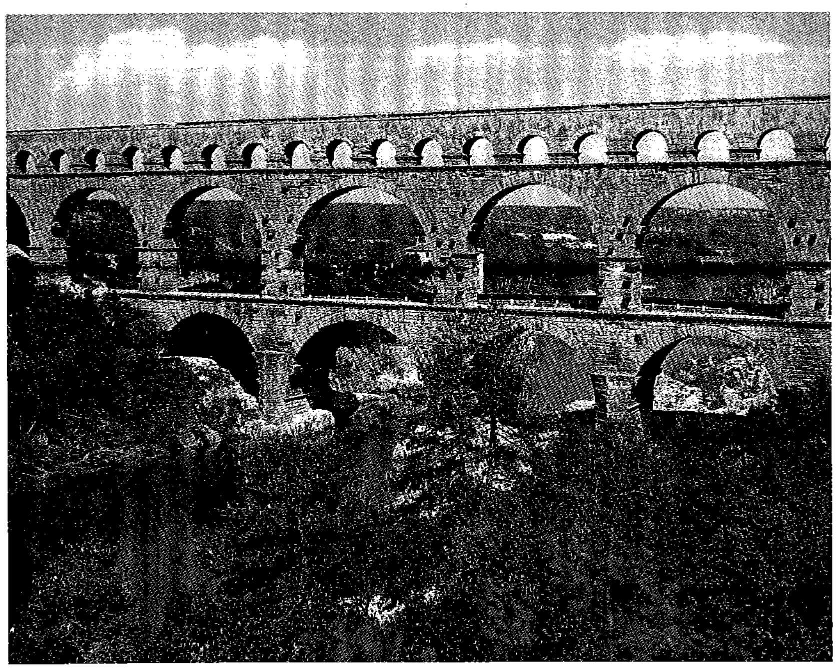


FIGURE 2.1

The Pont du Gard at Nîmes.

[1]Vitruvius, *The Ten Books of Architecture*, Bk. II, Ch. VI (New York: Dover, 1960), pp. 46–47.

2.2 HYDRAULIC LIMES

Both the Greeks and the Romans also produced hydraulic limes, by calcining limestones containing argillaceous (clayey) impurities. Moreover, they both knew that certain volcanic deposits, when finely ground and mixed with lime and sand, yielded mortars that were not only stronger than ordinary lime mortars, but were also resistant to water. The Greeks used a volcanic ash from the island of Santorin (Santorin earth). The Romans used a similar ash found around the Bay of Naples, which was generally called *pozzolana* because the best variety was found near the village of Pozzuoli, near Mt. Vesuvius. This material is mentioned by Vitruvius, writing in the second half of the first century B.C.:

> There is also a kind of powder which from natural causes produces astonishing results. It is found in the neighborhood of Baiea and in the country belonging to the towns round about Mt. Vesuvius. This substance, when mixed with lime and rubble, not only lends strength to buildings of other kinds, but even when piers of it are constructed in the sea, they set hard under water.

Thus, pozzolan-lime mortars were used extensively in hydraulic structures, such as linings for aqueducts, cisterns, piers and sea walls. In addition, Vitruvius knew that if natural pozzolanas were not available, a similar effect could be obtained by using finely crushed burnt brick. The Romans also used these hydraulic mortars to make a form of concrete. Perhaps the best preserved building of the ancient world, the Pantheon (Figure 2.2), dating from the second century A.D., was built largely of concrete. The dome, 44 m (141 ft) in diameter, was constructed by pouring concrete into ribbed sections and letting it harden.

The quality of cementing materials generally declined through the Middle Ages. The art of burning lime was almost lost, and ground tiles (or pozzolans) were not added. High-quality mortars did not appear again until after the fourteenth century A.D., at which time the use of pozzolans was reintroduced. It was not until the eighteenth century that work first began on trying to understand the nature of these cementing materials. In 1756, John Smeaton (who was, incidentally, the first person to style himself a "civil engineer") was commissioned to rebuild the Eddystone Lighthouse off the coast of Cornwall, England. Recognizing that the normal lime mortars then in general use would not withstand the action of the water, Smeaton carried out an extensive series of experiments with different limes and pozzolans. He found that the best limestones for use in mortars were those containing a high proportion of clayey material. This was the first recognition of the factors that control the formation of hydraulic lime. Eventually, Smeaton used a mortar prepared from a hydraulic lime mixed with pozzolan imported from Italy. The Eddystone Lighthouse, thus rebuilt (Figure 2.3), stood for 126 years before it was replaced with a more modern structure.

2.3 DEVELOPMENT OF PORTLAND CEMENT

After this pioneering work, a number of other discoveries followed quite rapidly. James Parker in England took out a patent in 1796 on a natural hydraulic cement (misleadingly called Roman cement), produced by calcining nodules of impure limestone

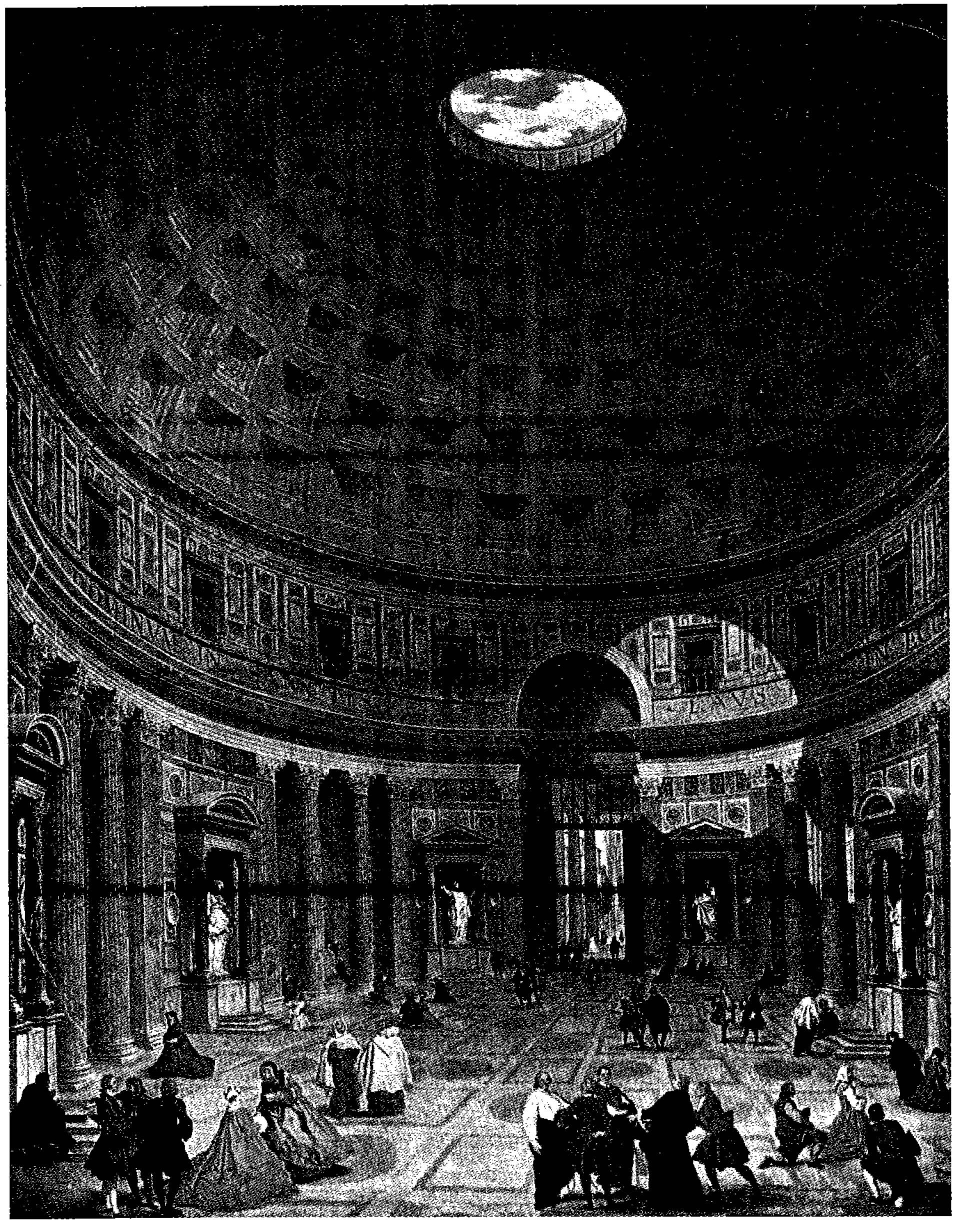


FIGURE 2.2

The interior of the Pantheon. (Giovanni Paolo Panini; National Gallery of Art, Washington, DC, Samuel H. Kress Collection.)

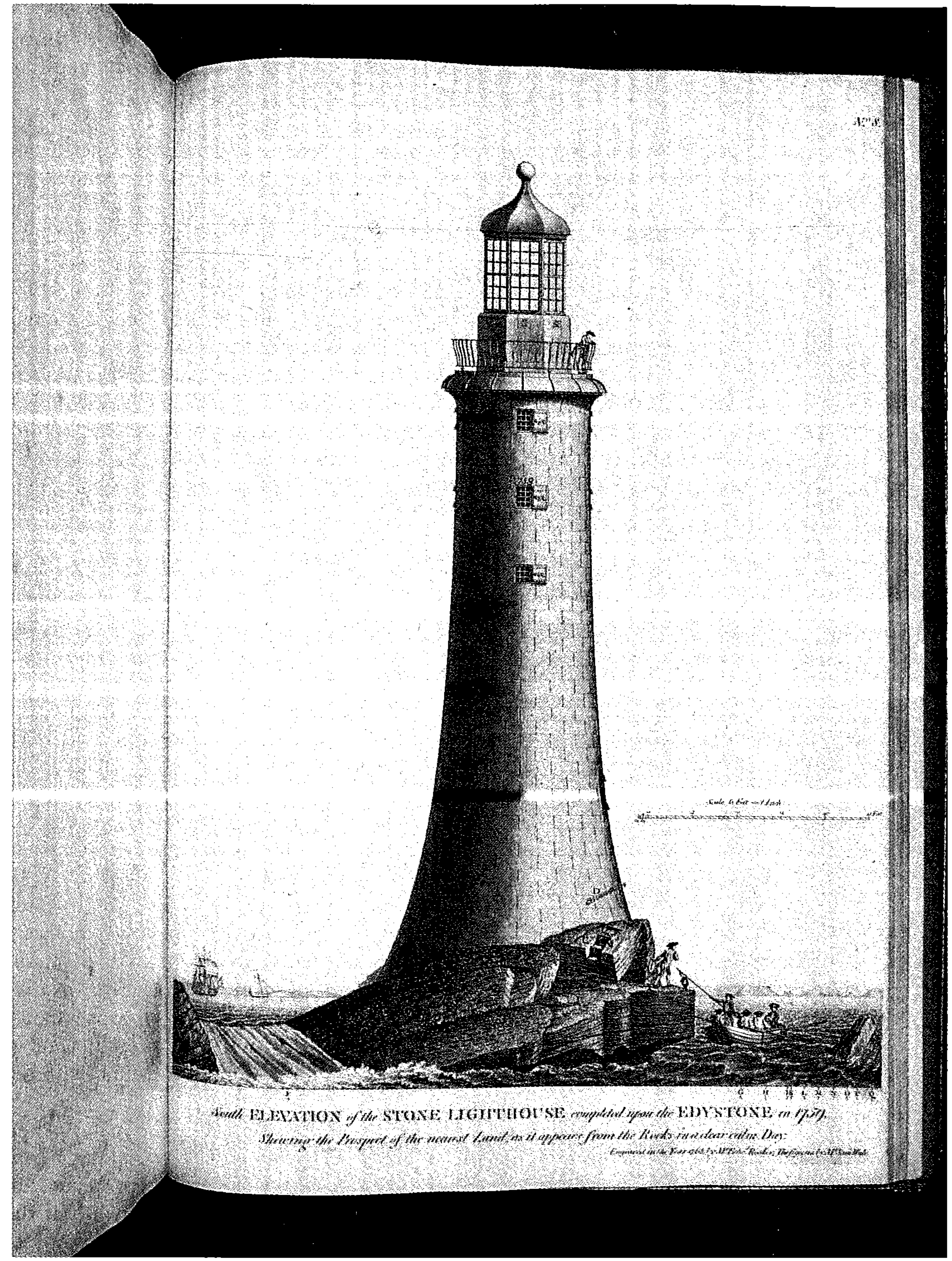


FIGURE 2.3

Smeaton's Eddystone lighthouse. (Illustration from John Smeaton, *A narrative of the building and a description of the construction of the Edystone lighthouse with stone*, London, 1791. Print courtesy of the Department of Special Collections, Kenneth Spencer Research Library, University of Kansas, Lawrence, KS.)

containing clay. A similar process began in France six years later. In 1813, also in France, Vicat (who developed the needles we still use to determine the time of setting of cement) prepared artificial hydraulic lime by calcining synthetic mixtures of limestone and clay. James Frost introduced the same approach in England in 1822.

Finally, in 1824, Joseph Aspdin, a Leeds builder, took out a patent on "portland" cement. The name "portland" was coined by Aspdin because of the real or fancied similarity of the hardened cement to a popular, naturally occurring limestone quarried at the Isle of Portland, which was used in many 18th and 19th century buildings in southern England. Aspdin's cement was prepared by calcining some finely ground limestone (taken, when available, "from the roads after it is reduced to a puddle or powder"), then mixing this with finely divided clay, and calcining the mixture in a kiln until the CO_2 was driven off. This mixture was then finely ground and used as cement. It might be noted that Aspdin tried to make his product as mysterious as possible by pretending to sprinkle secret salts into the kilns during burning. Whatever the exact nature of Aspdin's cement, however, he made it a commercial success.

It seems unlikely that true portland cement could be made according to Aspdin's patent; the temperatures were not high enough to cause real clinkering. Isaac Johnson claimed to have first burned the raw materials to the clinkering temperature in 1845 to produce modern Portland cement. Petrographic examinations of some structures from this period seem to support this claim.

The use of portland cements spread rapidly through Europe (due in no small part to Aspdin's son, William) and North America. Natural cements are made by burning an argillaceous limestone that has the right composition of lime and silica. However, the burning temperature is only high enough to cause partial chemical combination. Thus, natural cements are intermediate between hydraulic limes and portland cement. Although natural cement was produced in the United States as early as 1818, the first American patent on portland cement (by David Saylor) was not taken out until 1871. In Canada, limes and hydraulic cements were probably first produced in 1830. In the late nineteenth century considerable quantities of portland cement were imported by the United States from England and Europe. After domestic production was started, by about 1890, local portland cement quickly displaced the imports and captured the major construction markets from natural cement (Table 2.1).

Great advances were also made in the equipment necessary to produce portland cement. Originally, it had been made in vertical kilns. The disadvantage was obvious; the kiln had to be completely discharged before a new batch of cement could be made. The first rotary kiln was introduced in 1886, and soon Ransome in England was producing satisfactory kilns of this type. In the United States, Thomas Edison was issued a series of patents for rotary kilns in 1909, and for many years owned a successful cement company. It was in the United States also that gypsum was first interground with the clinker to increase the time of setting.

With the production of portland cement, work began on testing the material and characterizing its properties. Originally, tensile tests were carried out by building a cantilever beam of bricks held together by mortar and noting the length of beam that would support itself; this was taken to be proportional to the tensile strength of the mortar. The first systematic tests of tensile and compressive strength were carried out in Germany in 1836. After a great deal of experimentation in many countries, the basic

TABLE 2.1 Cement Production in the United States and Canada (Metric Tons).

	United States			*Canada*		
	Natural Cement	*Portland Cement*		*Natural Cement*	*Portland Cement*	
Year		*Imported*	*Domestic*		*Imported*[a]	*Domestic*
1880	450,000	18,000	4,500	–	1,600	–
1890	1,270,000	360,000	54,000	14,500	20,000	–
1900	1,810,000	270,000	1,540,000	20,000	60,000	46,000
1915	90,000	~ 900	14,100,000	–	4,500	910,000
1930	N.A.[b]	N.A.[b]	30,500,000	–	22,700	1,720,000
1950	540,000	41,000	42,100,000	–	227,000	2,630,000
1975	–	943,000	59,000,000	–	421,000	9,400,000
2000	–	21,200,000	88,800,000	–	628,000	12,600,000

[a]Both natural and portland cement
[b]N.A., not available

cement tests were largely standardized by about 1900. These tests, of course, have been modified since and new tests added as necessary.

2.4 CONCRETE ADMIXTURES

Historically, the use of admixtures is almost as old as concrete itself. It is known that the Romans used animal fat, milk, and blood to improve their concrete. Although these materials were probably added to improve the workability of the concrete, blood (due to hemoglobin) is a very effective air-entraining agent and might well have improved durability.

The systematic study of chemical admixtures began with the introduction of air-entraining agents in the 1930s. This discovery was accidental—one of the grinding aids used in cement production was beef tallow, and it was found that where tallow was used, the resulting concrete was much more resistant to freezing and thawing. The leakage of oil into the cement in older grinding units had a similar effect. Both the Greeks and the Romans used volcanic ash in their hydraulic limes. Nowadays, fly ash and blast furnace slag are used in a similar fashion to improve the performance of concrete. Most modern concretes contain one or more admixtures in their formulations.

2.5 SUMMARY

This has, necessarily, been a very brief and selective outline of the history of cement and concrete. Although a great deal has been learned about cement and concrete since Roman times, a great deal still remains to be discovered. Much of our current research was foreshadowed in the last century by the pioneering studies of Vicat, Michaelis, Le Châtelier, and others, who first began a systematic study of the basic composition and properties of cement, with a view to determining the underlying physical and chemical causes for cement behavior. It is hoped that this brief account of cement and concrete has shown that any real advances in their use were based on fundamental studies, whether by Vitruvius or by Smeaton. Only by continuing this scientific approach will we be able to improve the properties of these materials to meet the challenge of the future.

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CHAPTER 3

Cements

There is a wide variety of cements that are used to some extent in the construction and building industries, or to solve special engineering problems. The chemical compositions of these cements can be quite diverse, but by far the greatest amount of concrete used today is made with portland cements. This chapter will therefore discuss the composition of portland cements in considerable detail and describe the more specialized cements briefly. The name *portland* originated as a trade name and thus it gives no indication of composition or properties. The name now applies to a family of closely related cements that have an overall similarity of properties. Different portland cements can be made for particular applications by modifying certain properties, and later we examine how these modifications are achieved by suitable manipulation of the basic composition of portland cement.

3.1 MANUFACTURE OF PORTLAND CEMENT

In principle, the manufacture of portland cement is very simple and relies on the use of abundant raw materials. An intimate mixture, usually of limestone and clay, is heated in a kiln to 1400 to 1600°C (2550 to 2900°F), which is the temperature range in which the two materials interact chemically to form the calcium silicates. In practice, because of the large amounts of materials being processed and the high temperatures required, considerable attention must be paid to the various stages of processing (see Figure 3.1) if adequate quality control is to be maintained.

Raw Materials

High-quality cements require raw materials of adequate purity and uniform composition. Limestone (calcium carbonate) is the most common source of calcium oxide, although other forms of calcium carbonate, such as chalk, shell deposits, and calcareous muds, are used. The location of cement plants is most often determined by the availability of suitable high-purity limestone, since a satisfactory source of silica can usually be obtained within close proximity. Iron-bearing aluminosilicates are invariably used as the primary source of silica. Clays or silts are preferred since they are already in a finely divided state; but shales, schists, and other argillaceous rocks are also used.

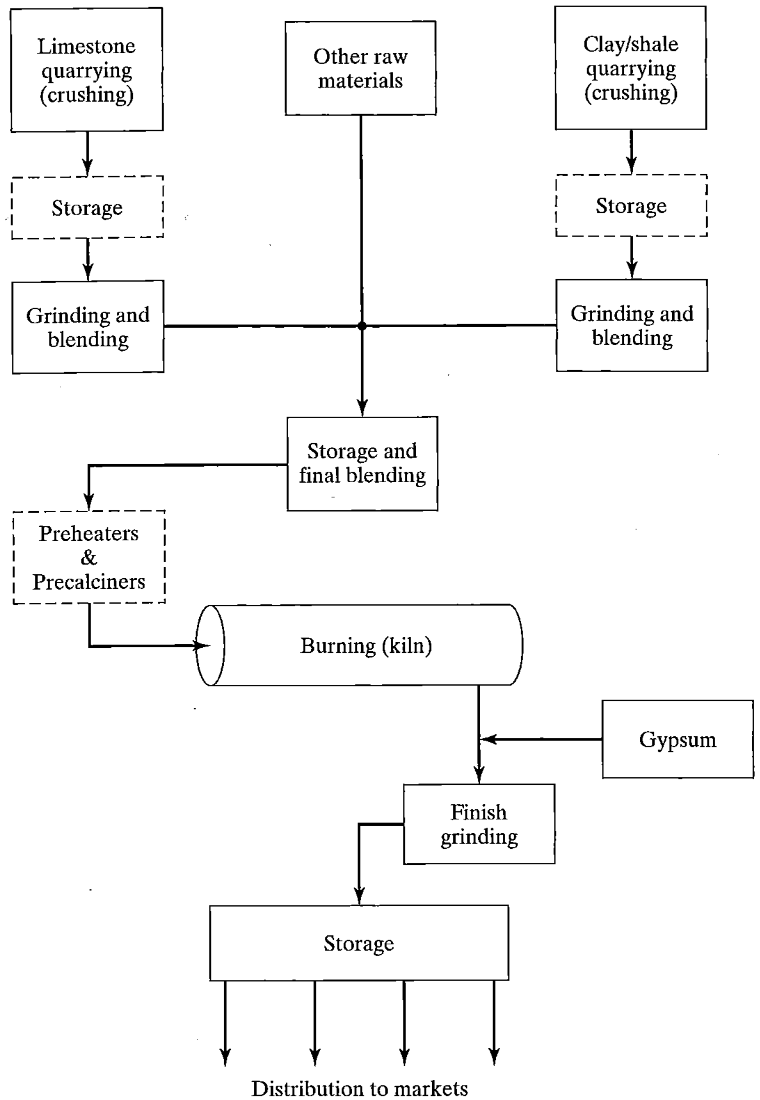


FIGURE 3.1

Schematic outline of cement production.

The compounds that are derived from the iron and alumina content of siliceous raw materials contribute little to portland cement as regards strength and can lead to problems of durability and abnormal setting behavior. It might be thought, therefore, that the use of pure silica would lead to a better cement. There are, however, good reasons for using aluminosilicates other than the scarcity of economic sources of pure silica. Quartz, the major form of pure silica in nature, is a relatively unreactive material and, moreover, pure lime–silica mixes have very high fusion temperatures (above 2000°C or 3600°F), so that reactions between the two components can only occur by a slow process of sintering. Aluminum and iron oxides act as *fluxing agents*, lowering the fusion temperature of a portion of the raw mix to a practical firing temperature. Quartz and iron oxide are quite commonly added (especially in the western United States) in small amounts to make up for deficiencies of SiO_2 and Fe_2O_3 in the main mixture.

Preparation of Materials

The object of processing the raw materials is to ensure that the raw feed entering the kilns is of constant composition and in a thoroughly pulverized and blended state. Failure to do this would result in a cement with variable composition and unpredictable properties. For example, if the composition of the raw feed varied widely, the percentages of the various chemical compounds in the finished cement would vary widely, making performance difficult to predict with certainty. The optimum burning temperature of the mix will also vary with composition. In addition, if the particle size is too large, complete chemical combination will not occur during the time the material is in the kiln and will result in a cement of inferior performance.

The exact sequence of operations at this stage may vary considerably from plant to plant, depending on the raw materials, equipment, and plant design. In the *wet process*, each raw material is quarried, crushed, and stockpiled separately to provide feedstocks of known composition. Blending of the materials may take place during fine grinding or after each material has been ground separately. For example, clay can be broken down to a finely divided slurry merely by good mechanical agitation. Wet grinding of limestone is carried out separately in other mills to form a second slurry. When slurries are used, blending and final proportioning can be handled very conveniently. However, evaporation of large amounts of water consumes a large quantity of heat in the kiln and leads to substantially higher costs. Thus, in the final stages of the wet process, the water content of the slurry may be considerably reduced.

If the source of silica is an argillaceous rock, mechanical grinding is required and the advantages of the wet process are lost. Improvements in grinding mills have made the wet process uneconomic and obsolete, and modern cement plants use dry grinding processes. Additional energy that may be required for dry grinding is more than compensated for by savings in energy in the kiln. But even in the *dry process*, water may sometimes be added to the raw feed to granulate it for easy handling and good contact in the kiln. The nature of the raw materials will determine whether the calcareous and argillaceous rocks are ground together or separately. In the dry process, care must be taken to ensure adequate mixing and blending and to avoid excessive loss of fines. Closed-circuit grinding with classifying equipment is commonly used to avoid overgrinding and control of dust. Ground materials are stored in silos, and final mixing may be accomplished during transfer to a final storage silo before reaching the kiln.

In some plants, the *semi-dry* process is used. In this case, the raw mix is prepared as in the dry process, but then 12 to 14% of water is added to form nodules, which are fed into the kiln by means of a moving grate. Alternatively, the wet slurry is dewatered prior to entering the kiln.

The Burning Process

Once the raw feed has been satisfactorily ground and blended, it is ready to enter the kiln. This heat treatment is termed *clinkering*, to distinguish it from *sintering* (where no melting occurs) and *fusion* (where complete melting occurs). In the cement kiln, partial melting takes place; only about one-fourth of the charge is in the liquid state at any time, but it is in this fraction that the necessary chemical reactions proceed. The kiln, in the form of a long steel cylinder lined with refractory brick inclined a few degrees from the horizontal, is rotated at about 60 to 200 rev/h about its axis. Older kilns may reach 6 m (20 ft) in diameter and over 180 m (600 ft) in length with a production capacity exceeding 5000 tons/day using the dry process. The raw feed enters at the high end, and the combination of rotation and inclination slowly moves the material the length of the kiln. In the wet process, the material may stay in the kiln for 2 to $2\frac{1}{2}$ h; this is reduced to 1 to $1\frac{1}{2}$ h for dry kilns and to as little as 20 min for some kilns with modern heat exchangers (suspension preheaters). In the latter case, kilns may be less than 80 m (250 ft) in length.

At the low (front) end of the kiln, a mixture of fuel and air is injected and burned with the exhaust gases passing through the kiln to the stack. The traditional fuel used is powdered coal, but oil and natural gas may be used depending on the price and availability. (Some companies use supplementary sources of fuel, such as old tires or waste solvents; the cement kiln is a very efficient combustor of organic materials.) The raw material therefore moves gradually into zones of increasing temperature. (see Figure 3.2). Four distinct processes take place in the kiln: evaporation, calcination, clinkering and cooling, and each stage is important to the overall burning of the raw feed. The exit gases at the back end of the kiln are at a temperature of 250 to 450°C and quickly raise the temperature of the feed to the point at which free water is lost by evaporation. In the wet process, the *evaporation zone* may require one-third to one-half of the total length and can be assisted by putting heat exchangers, such as chains, inside the kiln. Dry process kilns require less than one quarter of the length for evaporation, while modern plants use a *preheater tower* (see Figure 3.3). Here the feed is heated to calcination temperatures in a matter of a few seconds, the feed passing downward through a series of cyclones through which the hot gases are flowing.

Once free water is lost, the charge quickly heats up to calcination temperature, losing first water from the argillaceous materials at about 600°C and the carbon dioxide from the limestone at about 900°C. The *calcination zone* extends more than half the length of the kiln and transforms the charge into a reactive mixture of oxides that can enter into new chemical combinations. Since calcination takes place at relatively low temperatures, it can be accomplished in a special furnace that can burn lower cost fuels of lower calorific value. Such a furnace is placed between the preheater tower and the kiln and is called the *precalciner*. Initial chemical combination takes place in the latter part of the calcination zone around 1200°C, when calcium aluminates and ferrites form through solid-state reactions. These compounds act as *fluxes*, melting around 1350°C to begin the *clinkering zone*. Although it may occupy perhaps only one-fourth of the total length of a long kiln, it is the heart of the kiln where final chemical combination occurs to form the calcium silicates. The charge temperature rises to 1400 to 1600°C during the time (15 to 45 min) spent in this zone. In modern plants, only chemical combination and

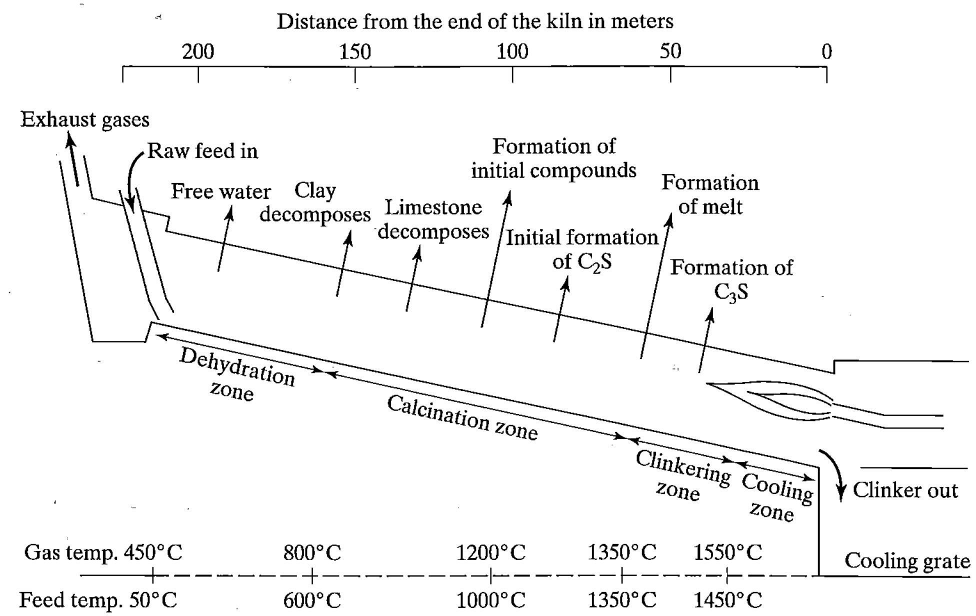


FIGURE 3.2

Schematic outline of conditions and reactions in a typical dry-process rotary kiln. When suspension preheaters are used, dehydration and initial calcination takes place outside the kiln in the preheater tower.

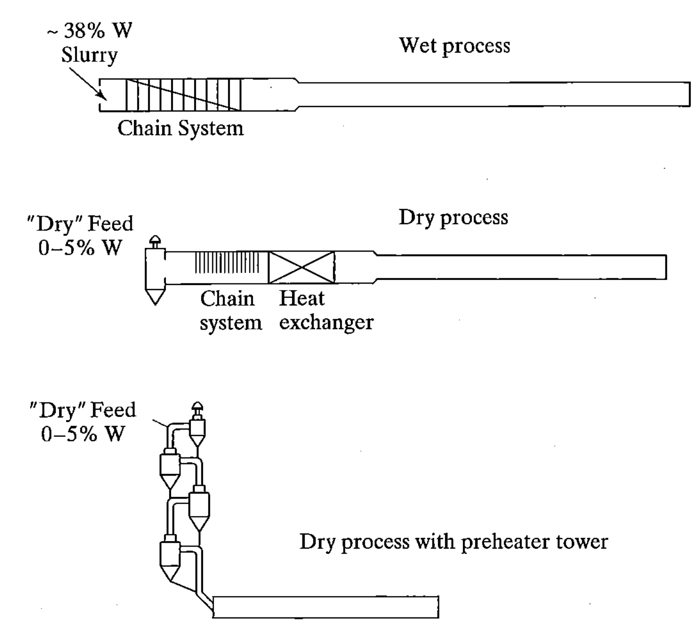


FIGURE 3.3

Comparison of kiln systems in older and modern cement plants. (W = water content of incoming feed.)

clinkering take place in the kiln, thus reducing kiln length to less than 100 m (330 ft), increasing throughput and lowering energy requirements.

As the charge moves past the flame in the final few meters, it rapidly drops off in temperature in the *cooling zone*. Here the liquid phase solidifies again to produce the hard nodules of clinker, which exit at a temperature of about 1000 to 1200°C. The rate at which the clinker cools can significantly alter the reactivity of the final cement.

Final Processing

The material that emerges from the kiln is known as *clinker*; further processing is still required. The clinker, in the form of dark-gray porous nodules [6 to 50 mm (0.25 to 2 in.) in diameter] is still hot and is further cooled by an air or water spray, typically on a moving grate. The clinker is conveyed to ball mills, where it is ground to the fine powder that is so familiar to us. A small amount of gypsum is interground with the clinker to control the early reactions of tricalcium aluminate that, without the addition of gypsum, can cause *flash setting* of the clinker. Portland cement is clinker interground with gypsum; without the gypsum, it is only ground clinker. The ground cement is subsequently stored in large silos until ready for distribution. At this stage, a final blending of the cement will improve the uniformity of performance of the product by averaging out small differences in chemical composition and burning conditions that inevitably occur during manufacture.

Quality Control

The manufacture of portland cement can be seen to involve complex chemical reactions, and all stages of production should be closely monitored and controlled. Plant chemists analyze the raw materials at all stages of production, as well as the finished products, and on-line automated analytical controls have been developed. The burning process is complex and not yet fully understood. Thus, although automated control of kilns is progressing, optimum operating conditions are still, by and large, based on trial-and-error experience and require manual supervision.

Economic Considerations

The manufacture of portland cement is an energy-intensive process, due primarily to the high temperatures that must be maintained in the rotary kilns (see Table 3.1). The wet process is energy-wasteful because of the extra heat required to vaporize the slurry water. The use of the preheater tower, which is a very efficient heat exchanger that not only dries the raw feed before it enters the rotary kiln, but may also partly calcine it, is more energy-efficient; it is being installed in modern plants. Coupled with other technological improvements, the average fuel consumption is 4.5 MJ/kg in Japan and 4.2 MJ/kg in Germany, compared to 5.4 MJ/kg (4.8×10^6 BTU/ton) in the United States. The use of preheaters together with precalciners can also increase the production capacity of a given kiln by 50% or more. A modern cement plant is one of the most energy efficient, large-scale thermal processing operations; the theoretical fuel requirement is about 1.7 MJ/kg (1.5×10^6 BTU/ton). In spite of their lower thermal efficiency, some wet process plants have maintained their economic viability because of the use of waste materials (solvents, old tires, etc.) as fuels. The cost of fuel is not only low, but may involve additional economic incentives, since the kiln serves as a disposal site for otherwise undesirable materials.

TABLE 3.1 Energy Consumption in the US Cement Industry, Average Values [MJ/kg(10^6 BTU/metric ton)][a]

	Wet Process	*Dry Process*	*All Plants*
Fuel Consumption	6.06 (5.74)	4.41 (4.18)	4.77 (4.52)
Power consumption[b]	142	156	153
Fuel + Power	6.56 (6.22)	4.97 (4.71)	5.32 (5.04)

[a] 1999 data taken from *U.S. Cement Industry Fact Sheet*, 2001 ed., Portland Cement Assoc., Skokie, IL.
[b] kWh/metric ton.

3.2 COMPOSITION OF PORTLAND CEMENT

Compound Composition

A typical chemical composition of a general-purpose (ordinary) portland cement that can be purchased at a local building supply store is given in Table 3.2. It will be noted that the quantities do not add up to 100%, the missing percentages being accounted for by impurities. Since the calcium silicates account for three-fourths of the cement and are responsible for its cementing qualities, it might be more logically called calcium silicate cement. The major difference between "natural" cements, with a similar chemical composition, produced in the 19th century and portland cements is the presence of tricalcium silicate, which requires the temperature in the kiln to reach 1400°C(2550°F).

The chemical formulas of these compounds are traditionally written in an oxide notation frequently used in ceramic chemistry. This gives rise to a unique, but useful, shorthand notation that has universal use among cement scientists. The common notation is summarized in Table 3.3. Each oxide formula is abbreviated to a single capital letter, with the number of oxide formulas in the compound designated by a subscript placed after the letter. To write carbonates and sulfates (which are important in cement chemistry) in the shorthand notation, it is necessary to make use of CO_2 and SO_3; the symbols $\overline{C}$ and $\overline{S}$ have been adopted. Thus, calcium carbonate ($CaCO_3$) is written as

TABLE 3.2 Typical Composition of Ordinary Portland Cement[a]

Chemical Name	*Chemical Formula*	*Shorthand Notation*	*Weight Percent*
Tricalcium silicate	$3CaO \cdot SiO_2$	C_3S	55
Dicalcium silicate	$2CaO \cdot SiO_2$	C_2S	18
Tricalcium aluminate	$3CaO \cdot Al_2O_3$	C_3A	10
Tetracalcium aluminoferrite	$4CaO \cdot Al_2O_3 \cdot Fe_2O_3$	C_4AF	8
Calcium sulfate dihydrate (gypsum)	$CaSO_4 \cdot 2H_2O$	$C\overline{S}H_2$	6

[a] See the section entitled "Specifications and Properties," however, concerning cements produced in the western states.

TABLE 3.3 Typical Oxide Composition of a General-Purpose Portland Cement

Oxide	Shorthand Notation	Common Name	Weight Percent
CaO	C	lime	64.67
SiO_2	S	silica	21.03
Al_2O_3	A	alumina	6.16
Fe_2O_3	F	ferric oxide	2.58
MgO	M	magnesia	2.62
K_2O	K	alkalis	0.61
Na_2O	N		0.34
SO_3	$\overline{S}$	sulfur trioxide	2.03
CO_2	$\overline{C}$	carbon dioxide	–
H_2O	H	water	–

$CaO \cdot CO_2$ or as $C\overline{C}$; gypsum ($CaSO_4 \cdot 2H_2O$) is written as $CaO \cdot SO_3 \cdot 2H_2O$ and becomes $C\overline{S}H_2$ in shorthand notation. Although it is possible to determine the *compound composition*, as given in Table 3.2, by direct analysis, the methods employed are complex and require special skills and expensive equipment. Therefore, it is usually estimated by calculation using the ideal compound stoichiometries and an oxide analysis determined by standard methods (usually x-ray fluorescence spectroscopy). Such analyses are available from the cement supplier upon request; a typical oxide analysis is given in Table 3.3.

The calculation of the phases from the composition is known as the *Bogue calculation* (after R. H. Bogue, who first emphasized the advantages of knowing the compound composition of a portland cement). A simple Bogue calculation, suitable for most purposes, is given in ASTM C 150, although more sophisticated procedures have been developed. The necessary equations are as follows (the quantities C_2S, A, F, C_3S, and so on, represent the percentages by weight of the various components):

Case A:

$$A/F \geq 0.64$$
$$C_3S = 4.071C - 7.600S - 6.718A - 1.430F - 2.852\overline{S}$$
$$C_2S = 2.867S - 0.7544C_3S$$
$$C_3A = 2.650A - 1.692F$$
$$C_4AF = 3.043F$$

Case B:*

$$A/F < 0.64$$
$$C_3S = 4.071C - 7.600S - 4.479A - 2.859F - 2.852\overline{S}$$
$$C_2S = 2.867S - 0.7544C_3S$$
$$C_3A = 0$$
$$C_4AF^* = 2.100A + 1.702F$$

* Note: In Case B, C_4AF has a different composition from that in Case A. C_2F may also be present and is included in the quantity calculated.

Applying the Bogue equations (Case A) to the oxide analysis in Table 3.3 gives the following compound composition: C_3S = 52.6; C_2S = 20.6; C_3A = 12.0; C_4AF = 7.9.

ASTM Classifications

A knowledge of the compound composition of portland cement makes it possible to predict the properties of the cement. More important, manipulation of compound composition can be used to modify certain properties of the cement so that the cement will perform more satisfactorily in particular applications. To understand and appreciate this, it is necessary to anticipate some material that will be discussed in more detail in Chapter 4.

Hydration of Cement When portland cement is mixed with water, its constituent compounds undergo a series of chemical reactions that are responsible for the eventual hardening of concrete. Reactions with water are designated *hydration*, and the new solids formed on hydration are collectively referred to as *hydration products*. All chemical reactions, including hydration, can be described by reaction stoichiometries, rates of reaction, and heats of reaction. Also, in the case of cement chemistry, it is of interest to know whether the hydration products contribute to the strength of the hydrated cement. The hydration characteristics of the cement compounds are summarized in Table 3.4. The stoichiometries are described by chemical equations and need not be discussed at this time.

The rates of hydration for the pure cement compounds are given in Figure 3.4a. It can be seen that C_3A and C_3S are the most reactive compounds, whereas C_2S reacts much more slowly. The presence of gypsum slows the early rate of hydration of C_3A. Quantitative data are not available for C_4AF, but the reaction of C_4AF + gypsum + water is believed to be somewhat slower than C_3S, whereas the hydration of C_4AF without gypsum is faster. The rate of hydration of the compounds as they occur in portland cement is plotted in Figure 3.4b, where it can be seen that alite (impure C_3S) and belite (impure C_2S) react more rapidly than they do as pure pastes, and C_4AF (ferrite) hydrates more slowly than C_3S. The actual rate of hydration depends on the particular cement. It is to be noted that both C_3A and C_4AF react with gypsum as well as water during cement hydration. Rates of reactions need not have a direct relation to strength development, however, as can be seen in Figure 3.5. Clearly, the calcium silicates provide

TABLE 3.4 Characteristics of Hydration of the Cement Compounds

Compounds	Reaction Rate	Amount of Heat Liberated	Contribution to Cement: Strength	Contribution to Cement: Heat Liberation
C_3S	Moderate	Moderate	High	High
C_2S	Slow	Low	Low initially, high later	Low
$C_3A + C\overline{S}H_2$	Fast	Very high	Low	Very high
$C_4AF + C\overline{S}H_2$	Moderate	Moderate	Low	Moderate

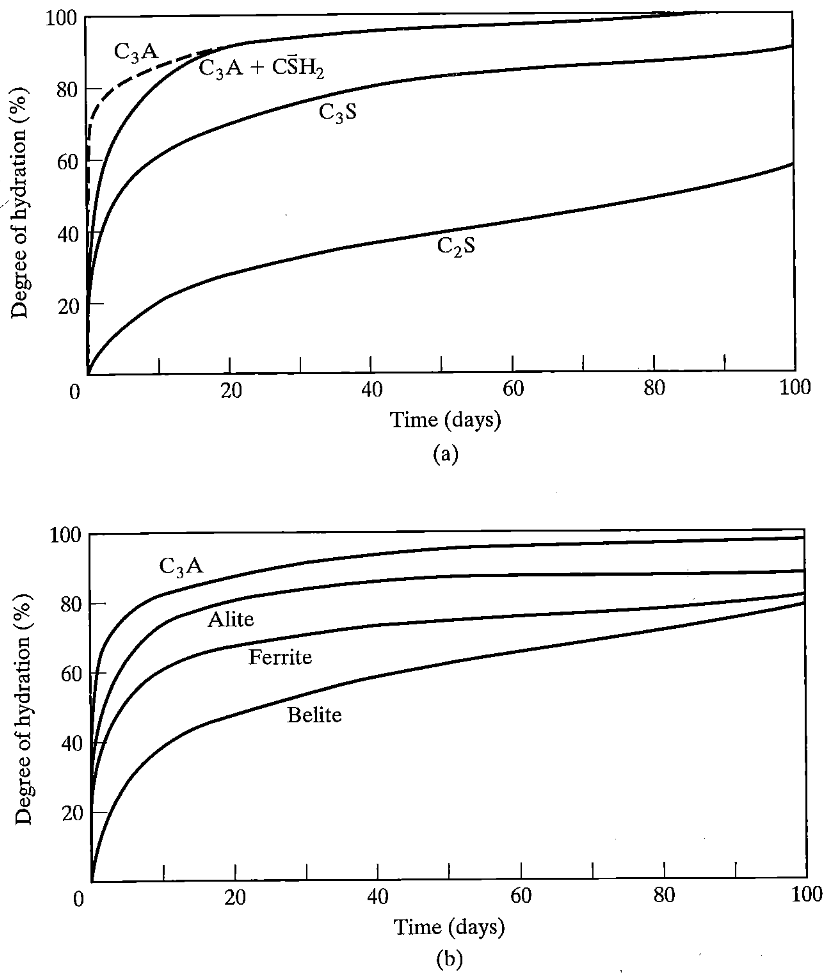


FIGURE 3.4

Rate of hydration of the cement compounds: (a) in pastes of the pure compounds; (b) in a Type I cement paste.

most of the strength developed by portland cement; C_3S provides most of the early strength (in the first three to four weeks); and both C_3S and C_2S contribute equally to ultimate strength.

The hydration reactions of portland cement are all *exothermic*; that is, they liberate heat. Thus, during the hardening process, the concrete is being continually warmed by the internal heat generated. The extent of temperature rise in a concrete section will depend on how quickly the heat is liberated and how quickly it is lost from the concrete to the surroundings. Therefore, the *rate* of heat evolution is the important quantity. The contribution of each compound to the overall rate of heat evolution is a

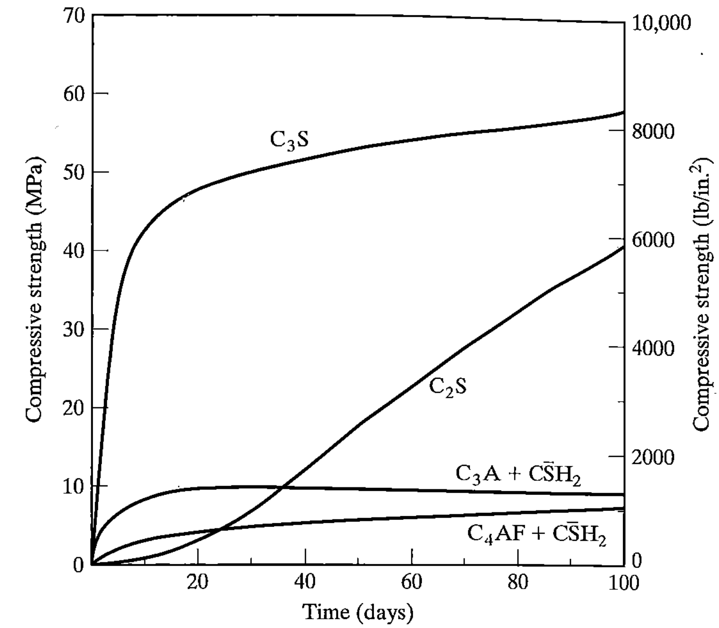


FIGURE 3.5

Compressive strength development in pastes of the pure cement compounds.

function of the heat of hydration, the rate of hydration, and the fraction of the compound in the cement. Multiple regression analyses give the following relationships for the heat of hydration in J/g of a typical cement:

$$H_{3\,\text{days}} = 240(C_3S) + 50(C_2S) + 880(C_3A) + 290(C_4AF) \qquad (3.1a)$$

$$H_{1\,\text{year}} = 490(C_3S) + 225(C_2S) + 1160(C_3A) + 375(C_4AF) \qquad (3.1b)$$

The quantities of C_3S, C_2S, and so on, are expressed as weight fractions of the cement. It is to be noted that the coefficients in Eq. (3.1b) are similar to the heats of hydration for complete hydration of the pure compounds (see Table 4.3). Using the compound composition calculated earlier from Table 3.3, the values of $H_{3\,\text{days}}$ and $H_{1\,\text{year}}$ are 265 J/g and 473 J/g, respectively.

ASTM Types Once the general role of the cement compounds is known, it should be possible, by suitable adjustment of compound composition, to modify the properties of portland cements. This is now done commercially in most countries. In the United States, five distinct portland cements are recognized by ASTM, differing only in the relative amounts of the cement compounds and the degree of fineness. These cements are generally known by their ASTM designation or the equivalent description; typical compositions for these cements are given in Table 3.5. The Canadian Standards Association (CSA) has a similar designation.

ASTM Type I is the cement most commonly used in general construction where no special properties are needed or specified. If, however, a more rapid rate of strength gain is desirable, as, for example, in precast work where forms are reused or when concreting at low temperatures, a Type III cement can be specified. Increasing the C_3S

TABLE 3.5 Typical Chemical Composition and Properties of Portland Cements, ASTM Types I to V

	I^a	II	III	IV	V
C_3S	55	55	55	42	55
C_2S	18	19	17	32	22
C_3A	10	6	10	4	4
C_4AF	8	11	8	15	12
$C\bar{S}H_2$	6	5	6	4	4
Fineness (Blaine, m^2/kg)	365	375	550	340	380
Compressive strength[b] [1 day, MPa (lb/in.^{2})]	15 (2200)	14 (2000)	24 (3500)	4 (600)	12 (1750)
Heat of hydration (7 days, J/g)	350	265	370	235	310

[a] CSA designations are 10, 20, 30, 40, and 50, respectively.
[b] ASTM C 109 50 mm mortar cubes.

content will achieve this aim, but grinding the cement more finely is even more effective. The resultant increased surface area that will be in contact with water means faster hydration and more rapid development of strength. The amount of strength gain of a Type III cement over the first 24 h is about double that of Type I.

The high C_3S content and rapid rate of hydration result in a high rate of heat evolution. If this heat is not allowed to escape (i.e., if *adiabatic conditions* exist), the temperature rise can be quite large as shown in Figure 3.6. In mass concrete, true adiabatic conditions can be approached when concrete volumes are large in comparison to the surface from which the heat can be dissipated to the surroundings. The generation of high internal temperatures occurs primarily over the first few days, when the concrete is still relatively plastic, and will not cause immediate damage. However, trouble may occur during subsequent cooling when the concrete has gained rigidity, since the resulting thermal stresses may cause tensile cracking. For this reason, Type III cement should not be used when placing thick [> 0.5 m (1.6 ft)] concrete sections such as piers and abutments. Even the more modest rate of heat evolution for Type I may cause problems when the heat is not rapidly dissipated or when cement contents are high. Temperature rises approaching 30°C have been measured within massive placements.

Thermal cracking was a frequent problem during the construction of some of the earlier concrete dams, even though low cement contents were generally used. This problem was largely solved by the development of ASTM Type IV, low heat of hydration cement. Since C_3S and C_3A are responsible for most of the early liberation of heat, reduction in the amounts of these compounds substantially reduces the amount of heat produced (see Figure 3.6). With the use of Type IV cements, thermal cracking can be eliminated provided that reasonable care is taken in controlling the initial temperature of the concrete and ensuring adequate opportunity for heat dissipation. In mass concrete applications, the slow rate of hardening that results from low C_3S contents is of little consequence.

Another problem that was troublesome during the earlier days of concrete construction was the deterioration of portland cement concrete when exposed to waters or soils containing sulfates. An obvious example of such an aggressive medium is seawater,

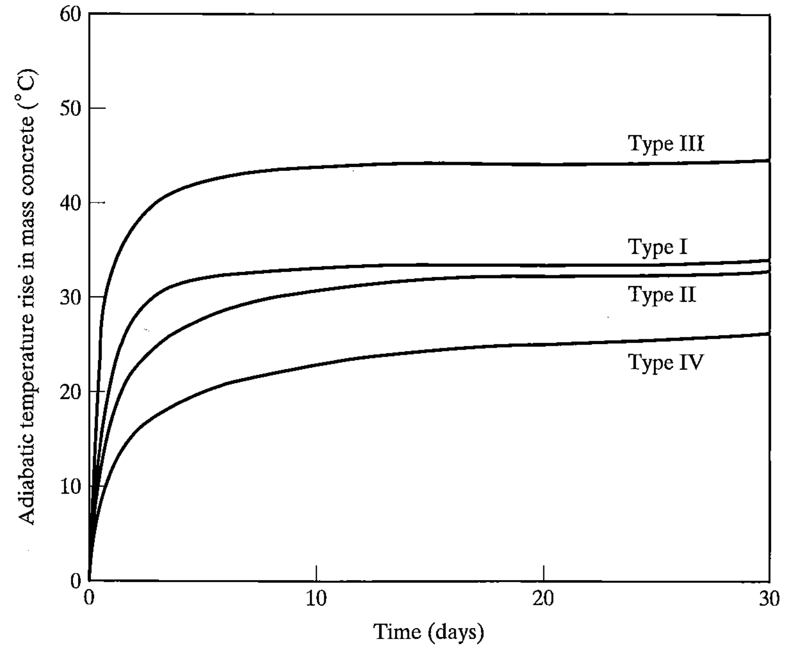


FIGURE 3.6

Rate of heat evolution measured as a rise in temperature of mass concrete stored under adiabatic conditions. [Based on data in *Concrete Manual*, 8th ed., Bureau of Reclamation, Denver, CO (1975).]

SO_3 attack

particularly when a structure is in the intertidal zone and is also exposed to wetting and drying. But many groundwaters contain high concentrations of sulfate and are more aggressive than seawater under conditions of total immersion. Sulfate attack involves interactions between the hydration products formed from C_3A, as will be discussed more fully in Chapter 4. Lowering the C_3A content by converting it to C_4AF, which is not so susceptible, has been found to be an effective means of combating sulfate attack. Experience has shown that keeping the C_3A content below 5% by weight, as is done in a Type V cement, will provide satisfactory performance in the most aggressive media (see Table 10.7 in Section 10.3) provided that the water-to-cement ratio is low as well (see Chapter 18). Somewhat higher amounts of C_3A, up to 8% by weight, can be tolerated for lesser amounts of sulfate. Type IV cement is a moderately sulfate-resistant cement. However, because its low rate of strength development is a disadvantage, Type II cement was developed. This cement has a higher C_3S content than Type IV, and therefore better strength development. But it has a higher heat of hydration. It is similar in performance to Type V except for its lower sulfate resistance. Type II is also often used when a lower heat of hydration is required, but with a more normal rate of strength gain.

Specifications and Properties It should be noted that the compositions for the various ASTM types given in Table 3.5 are only representative values and that actual

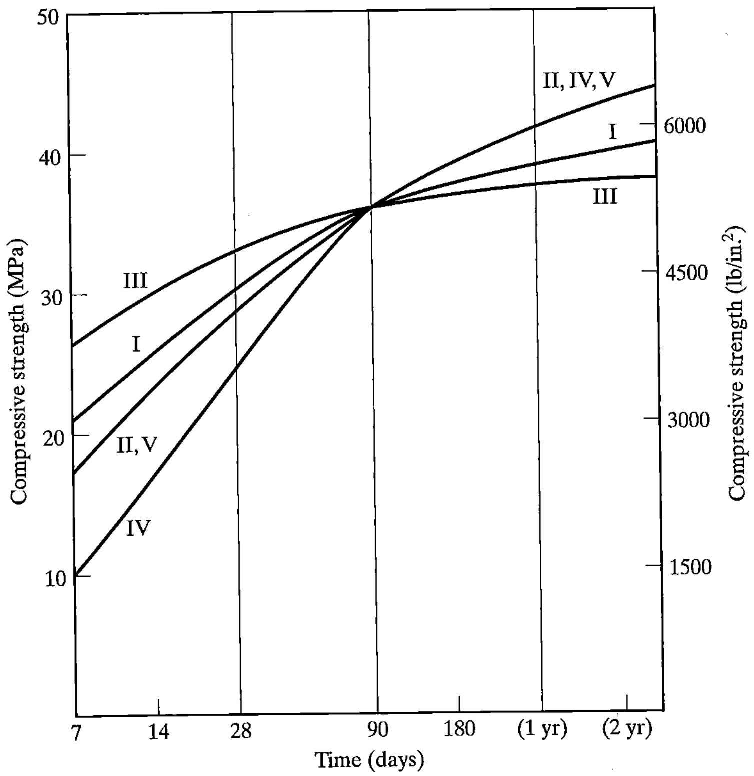


FIGURE 3.7

Strength of 6 × 2 in. (150 × 300 mm) concrete cylinders made with the same aggregate, but different cements. [Adapted from *Concrete Manual*, 8th ed., Bureau of Reclamation, Denver, CO (1975).]

compositions can vary widely within each type. For example, although Types I and III cement frequently have similar compositions, with Type III ground more finely, some cement companies make a Type III cement with enhanced C_3S contents of 60% or higher. Although ASTM specifications (see Section 3.5) list chemical requirements for all types, it will be seen (Table 3.9) that there are few limitations on compound composition. ASTM specifications are in essence performance specifications that ensure that certain physical requirements (given in Table 3.10) are met regardless of actual chemical composition. Other national standards have adopted the same philosophy and allow considerable latitude in chemical composition. It should also be remembered that the existence of an ASTM specification does not guarantee the commercial availability of a particular cement type. The manufacture of special types of cement requires a change in raw feed composition and burning conditions and the provision of extra storage facilities. Cement companies will not invest the time and money required for the production of a different cement unless an adequate market exists to allow them to recoup costs. Thus, only Types I and III are available throughout the United States; Type V is only produced in localities where special markets exist; while Type IV is

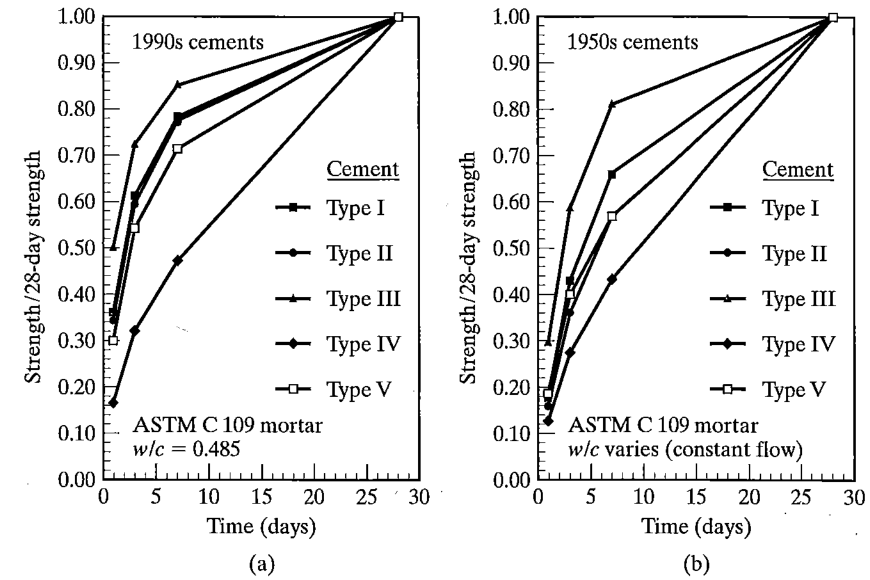


FIGURE 3.8

Average strength gain curves for portland cements manufactured in (a) the 1990s and (b) the 1950s. [From *Concrete Technology Today*, Portland Cement Assoc., Skokie, IL, (July 1996).]

rarely manufactured. Type II, or more likely Type "I/II" (which meets specifications for both types), is in common use in some parts of the country (e.g., the western states), where it is the common cement type rather than straight Type I. This is dictated by the composition of the most readily available raw materials. Where the properties of a special cement are needed but unobtainable, the performance of Type I can often be modified by the use of admixtures, as will be discussed in Chapters 5 and 8. A new performance-based specification ASTM C 1157, developed to cover portland cements blended with mineral admixtures, can also be applied to pure portland cements. This specification is covered in Chapter 5.

The ASTM types were developed to control particular aspects of cement hydration, but since all contain the same compounds, although in different proportions, it might be expected that the general properties of the hardened concrete would be very similar; and this is indeed the case. Furthermore, in all five types the sum total of the C_3S and C_2S contents is about the same: 75% by weight. Thus, the ultimate strengths of concretes and mortars should be similar even though the initial rates of strength gain are quite different. This is seen to be the case in Figure 3.7, although those cements that gain strength more slowly generally have higher long-term strengths.

It is worthy of note that the properties of the five standard types of portland cement have not been constant over time, but have varied with changes in the governing specification, ASTM C 150, and with the needs of the construction community. These changes can be illustrated by comparing cements manufactured in the 1990s with those manufactured in the 1950s. With the exception of Type III cement, cements manufactured

in the 1990s had higher average C_3S contents than cements manufactured in the 1950s. And in the 1990s, all five standard types were ground more finely than in the 1950s, *especially* Type III.

These changes affect the strength gain and setting times. The times to initial and final set were uniformly shorter in the 1990s than in the 1950s. Cements manufactured in the 1990s attained a greater portion of their 28-day strength during the first seven days than did the cements manufactured in the 1950s, as shown in Figure 3.8.

Impurity Oxides In Table 3.3 a number of impurity oxides were listed. Some, such as MgO, Na_2O, and K_2O, can be of considerable significance with regard to cement performance. Smaller amounts of other oxides, such as Mn_2O_3, P_2O_5, and TiO_2, are often found, but seldom significantly influence cement behavior.

It should be said at the outset that only approximate chemical equilibrium is attained in the rotary kiln and it is thus to be expected that all cement compounds will contain small amounts of the other oxides present in the clinker. Special analyses have indeed confirmed that this is so. The calcium silicates probably contain about 3% by weight of impurity oxides, principally Al_2O_3, Fe_2O_3, and MgO. Impure C_3S, as it exists in portland cement, is known as *alite* and impure C_2S as *belite*. As previously noted, both alite and belite are more reactive than the pure silicates and hydrate more rapidly (Figure 3.4). C_3A contains considerable amounts (about 10% by weight) of SiO_2 and Fe_2O_3, while C_4AF contains considerable SiO_2 and also much MgO. In fact, it is the presence of MgO in C_4AF that gives portland cement its familiar gray color; pure C_4AF is a chocolate brown color, but becomes black when MgO is present. Cements with low MgO contents may have a distinct brownish hue.

Even when free of impurities, however, C_4AF does not have an exact stoichiometry as do the other cement compounds. It is found that C_4AF is the compositional midpoint of a solid solution series between the end members C_2F and (hypothetical) C_2A. In most portland cements, the composition lies in the range C_6A_2F to C_6AF_2 and seldom will occur exactly as C_4AF. For this reason, it is often called the *ferrite phase* or *ferrite solid solution*, to avoid the implication of a fixed composition. The reactivity of the ferrite phase decreases with increasing iron content. Iron-rich ferrite phases are to be found in cements with a high ferrite content, such as Type V; some Type V cements may contain C_2F in addition to the ferrite phase.

It has been found from experience that limitations on the MgO content of the cement are required; otherwise, the mortar or concrete will start to expand and eventually crack some time after hardening. This is known as *unsoundness* and has been found to be caused by the fact that much of the MgO is present as crystalline MgO (periclase). Ordinarily, MgO hydrates quite rapidly to $Mg(OH)_2$. However, because it has been subjected to high temperatures in the kiln, it is much less reactive (dead-burned), and hydration takes place after the cement paste has hardened. The reaction

$$M + H \rightarrow MH \qquad (3.2)$$

involves an expansion in solid volume, and hence the periclase particles act as expansive centers that cause stresses within the matrix and eventual cracking.

Even if the MgO content is within specifications (less than 6%), unsoundness may develop if the crystals of periclase in the clinker are large. Thus, the cement must also pass the autoclave expansion test, which accelerates the potentially deleterious expansion by curing at a high temperature. This test also assesses the effect of *free lime* (uncombined CaO), which can cause expansion in an analogous manner. Although not specifically limited in the ASTM specifications, free lime contents are generally less than 0.5% in most U.S. cements. Values greater than 1.5% indicate poor control of the burning process and the likelihood of overall inferior performance, apart from unsoundness.

Another form of unsoundness is that due to bulk expansions resulting from high gypsum levels, which can reduce the strength of the paste. This is due to the formation of ettringite (discussed in Chapter 4); hence, the total SO_3 content is limited in ASTM specifications. There is an optimum addition of gypsum for each cement that will ensure maximum strength and minimum drying shrinkage. Most optimum gypsum levels are less than the equivalent amount of SO_3 permitted by ASTM. However, with some clinkers that have high sulfate contents, observance of the ASTM limit may preclude the addition of enough gypsum to reach the optimum level, since the ASTM specification does not distinguish between different origins of SO_3.

The presence of *alkalis* (Na_2O and K_2O) generally does not cause any special problems except when certain aggregates are used that can participate in the alkali–aggregate reaction (see Chapter 7). Under such circumstances, a low-alkali cement should be used (see Table 3.9). It is increasingly difficult, however, to produce low-alkali cements economically. Not only are sources of low-alkali raw materials becoming more scarce, but also modern processes tend to concentrate the alkalis in the clinker. Much of the alkalis in the raw feed volatilizes, and in older plants it was vented up the stack. (The alkalis recondense as the stack gases cool and now must be removed to comply with Environmental Protection Agency regulations.) In kilns with preheaters, the alkalis condense as the gases are circulated through the incoming feed, thereby increasing its alkali content or clogging up the preheater ducts or vessels. The use of high-sulfur coals also increases the amount of alkalis in the cement. Most of the additional alkalis are in the form of alkali sulfates (*free alkalis*) rather than as impurities in the cement compounds. These high-alkali cements often have higher rates of early strength development and can be prone to setting problems.

Limits are also placed on the insoluble residue that remains when portland cement is dissolved in concentrated hydrochloric acid. This is to avoid excessive dilution with inert materials. Insoluble residues are derived mostly from the added gypsum, although a part may be unreacted silica. The limit in ASTM C 150 is 0.75%.

3.3 MODIFIED PORTLAND CEMENTS

The five ASTM types just discussed all contain the four major clinker compounds—C_3S, C_2S, C_3A, and C_4AF—in varying proportions. There are a number of other portland cements that are formed either by adding other materials to the clinker or by forming other compounds during burning. These cements are collectively referred to as modified portland cements, since they are basically calcium silicate cements.

Blended Cements

Portland cements can be blended with pozzolans or blast furnace slag to achieve most of the characteristics shown by the five ASTM types. They can also be used to enhance durability and other properties not controlled by the five types. Collectively, such cements are known as blended cements and are covered by a separate specification: ASTM C 595. These will be addressed in more detail in the discussion of mineral admixtures (Chapter 5). In the United States, blended cements are relatively uncommon, since mineral admixtures are usually added at the concrete mixer. However, in contrast, almost all European portland cements are in fact blended cements.

Expansive Cements

One of the major disadvantages of portland cement concrete is the volume contraction that takes place on drying (shrinkage) and its susceptibility to tensile cracking if this contraction is wholly or partially restrained. Since random cracking is unsightly and can compromise the integrity of a structure, control of shrinkage cracking must be allowed for in design and construction. Cracking is particularly critical in water-retaining structures, or when entry of water must be prevented. Volume expansion during early hydration and hardening could be used to offset shrinkage, as shown schematically in Figure 3.9a. Although ordinary portland cements have very small expansion during moist curing, they can be modified to enhance early expansion, which can then be used to control shrinkage, as shown in Figure 3.9b. These cements were first developed in the United States, and commercial production began in the late 1960s. Since they are primarily used to offset the effects of drying shrinkage, they are commonly referred to as *shrinkage-compensating cements*.

Composition Hydraulic expansive cements must comply with the requirements of ASTM C 845, Type E-1. All cements under this classification are based on the formation of ettringite in considerable quantities during the first week of hydration. Three variants, K, M and S, are produced, depending on the nature of the aluminate compound used to generate ettringite:

$$\text{calcium aluminate} + \bar{S} + H \rightarrow \text{ettringite} \qquad (3.3)$$

The calcium aluminate replaces the C_3A in the cement and the calcium silicates still control long-term properties.

Type E-1(K) is the one used exclusively in the United States. It is composed of calcium sulfoaluminate ($C_4A_3\bar{S}$) with anhydrite ($C\bar{S}$), together with free lime, to enhance the rate of ettringite formation, and hence expansion. Type M uses monocalcium aluminate (CA) with gypsum, while Type S has a high content of C_3A (typically 20 wt. %). In principle, any reaction that causes a large bulk expansion could form the basis of an expansive cement, and formulations based on MgO or CaO have been investigated. A commercial lime-based cement is produced in Japan. The preferred method of producing an expansive cement is to add the expansive component to a regular portland cement.

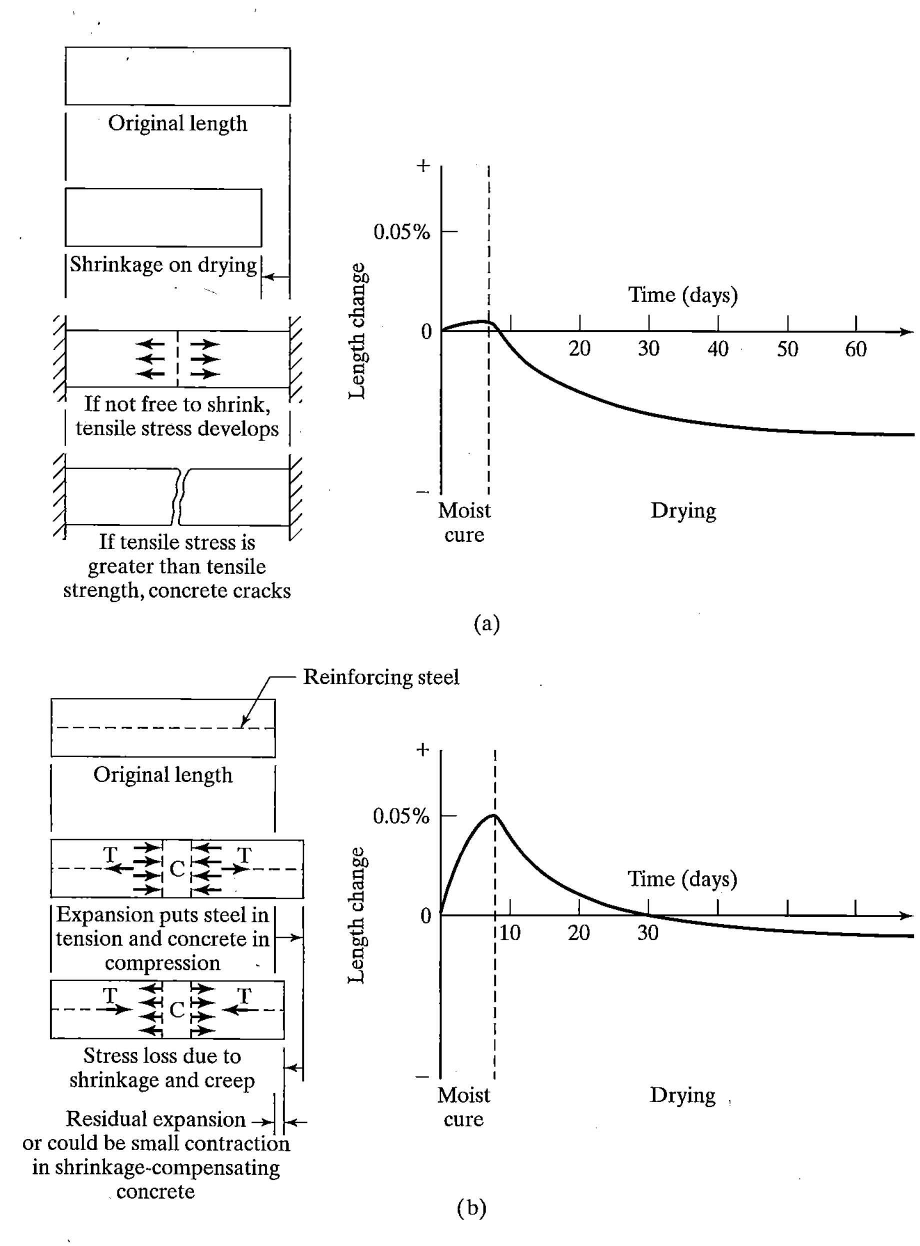


FIGURE 3.9

Drying shrinkage of concretes made with (a) Type I portland cement and (b) expansive cement concrete. [Adapted from M. Polivka and C. Willson, *Klein Symposium on Expansive Cement Concretes*, SP-38, ed. S. P. Shah, American Concrete Institute, Farmington Hills, MI, p.235 (1973).]

Mechanism of Expansion The potential expansion associated with ettringite formation is controlled by the use of ordinary steel reinforcement. The steel restrains the overall bulk expansion of the cement, thereby converting expansion into a slight prestress within the concrete. By resisting expansive forces, the steel is placed into tension and the concrete into compression. About 170 to 700 kPa (25 to 100 lb/in.^{2}) compressive stress is generated, which is generally sufficient to ensure that drying shrinkage will not cause tensile cracking: the concrete either remains in compression or develops only small tensile stresses. Some restraint of expansion may also occur through subgrade friction in slabs or from formwork. The exact prestress will depend on the amount of expansive components and the amount of reinforcement (Figure 3.10). Lack of sufficient restraint will provide little protection against shrinkage cracking and may, in extreme cases, cause the concrete to literally self-destruct because of excessive internal expansion. On the other hand, beyond a certain point, additional restraint becomes wasteful. Prestress is developed only in the direction and vicinity of the reinforcing steel. Thus, correct positioning of the steel is important to provide correct restraint, and misplaced reinforcement can lead to lack of adequate prestress or complications, such as warping due to differential expansions.

The amount of expansion before drying depends on the amount of expansive component, which should be chosen to ensure that the concrete will remain under a slight compression even after extended drying. Some expansion should be allowed to occur in order to develop maximum residual prestress. It should be emphasized that the development of adequate prestress is the key to shrinkage compensation; as can be seen in Figure 3.9b, concretes made with expansive cements shrink as much, if not more, than ordinary portland cement concretes. However, the net tensile stress generated is much less because of the initial compressive prestress.

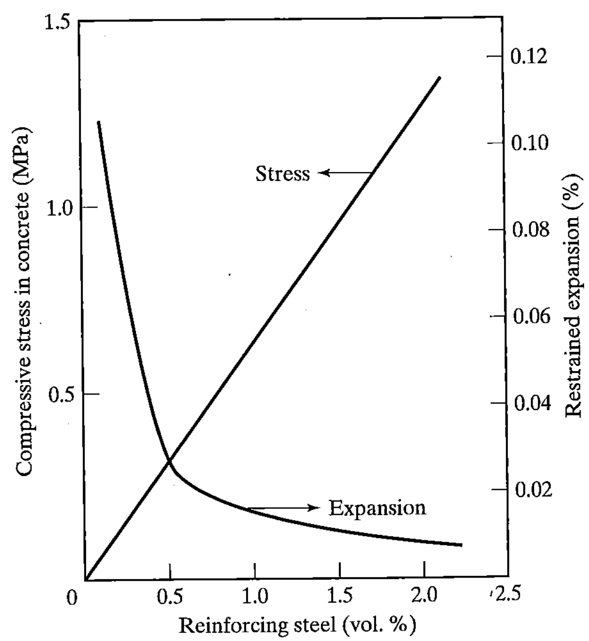


FIGURE 3.10

Effect of reinforcing steel on expansion of concrete and prestress developed. [Adapted from *New Materials in Concrete Construction*, University of Illinois at Chicago, p. 13-II (1972).]

TABLE 3.6 Variables Affecting Expansion in Expansive Cements

General Category	*Specific Variable*
Reinforcement	Amount of steel; positioning of rebars
Mix design	Cement content; water/cement ratio; fineness of cement; admixtures
Handling and curing	Time of mixing, handling, and placing; time of moist curing; temperature of curing

Control of Expansion Successful use of expansive cements depends upon proper control of the expansion of the cement during hydration and is sensitive to a number of variables (Table 3.6).

The amount of ettringite formed is controlled by adjusting the cement content or varying the amount of expansive admixture (if one is being used). This assumes that adequate moist curing is provided during the time when ettringite is forming—during the first seven days. Moist curing is even more critical than it is for ordinary portland cement concrete and requires the supply of additional water for hydration during the curing period, because the formation of ettringite combines large amounts of water, which cannot be completely provided by the mixing water. This is well illustrated in Figure 3.11, which compares the efficiencies of various standard curing procedures (described in Chapter 12). An increase in water/cement (w/c) ratio decreases the amount of expansion.

The other factors influencing expansion affect the amount of *useful* ettringite that is formed. Ettringite can only contribute to an overall volume expansion, and hence to prestress, when it is formed after the concrete has gained rigidity through hydration of C_3S. Any ettringite that forms while the concrete is still plastic is of little value. The effect of temperature on expansion illustrated in Figure 3.12 is an example. At 38°C hydration proceeds faster than at 21°C. Thus, the rate of expansion is faster, but ultimate expansion is lower because more ettringite is formed when the concrete is still plastic. The reduction in expansion that occurs with an increase in cement fineness is also a result of the faster initial rate of hydration. Admixtures may also interfere with the initial formation of ettringite and hence expansion, the effect of a particular admixture generally depending on whether it retards or accelerates set. Tests should be run when admixtures are to be used.

Properties The physical and engineering properties of expansive cement concretes are comparable to Type I cement concretes. Thus, in the absence of any specific data, the usual empirical relationships regarding strength, creep, shrinkage, and so on, may be used. Some differences have been observed in (1) compressive strength, (2) rate of heat evolution, (3) slump loss, and (4) resistance to sulfate attack. Concretes made with Type E-1(K) cement may have compressive strengths 3.5 to 7.0 MPa (500 to 1000 lb/in.^{2}) higher than comparable Type I cement concrete, particularly at lower w/c ratios and higher cement contents. This has been attributed to the formation of the large quantities of ettringite. But it could also be due to a more uniform microstructure

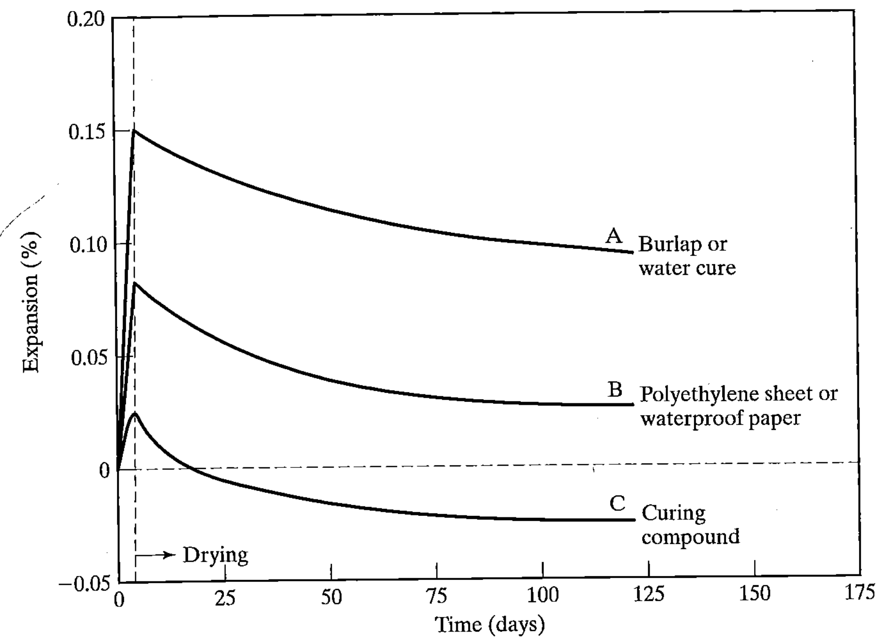


FIGURE 3.11

Effect of different curing procedures on the expansion of expansive cement concrete. [Adapted from C. E. Kesler, *Journal of the Construction Division*, ASCE, Vol. 102, No. CO1, pp. 41–49 (1976).]

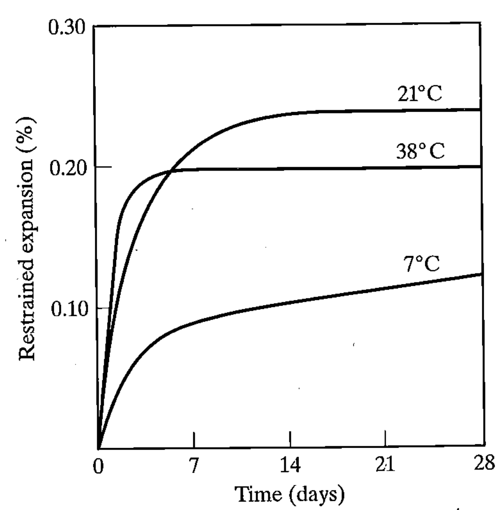


FIGURE 3.12

Effect of curing temperature on expansion during moist curing of Type E-IK cement concrete. [Adapted from M. Polivka in *Klein Symposium on Expansive Cement Concretes*, SP-38, ed. S. P. Shah, American Concrete Institute, Farmington Hills, MI, p. 250 (1973).]

of the hydrated paste since bleeding (see Chapter 9) does not occur. Reduction in bleeding is due to the early crystallization of ettringite, and this can also result in greater slump loss and higher water requirements. Expansive cements are generally not sulfate-resistant because of their high aluminate contents. However, certain Type E-1(K) cements with high $C_4A_3\overline{S}$ and low C_3A contents may be sulfate-resistant since ettringite remains the stable phase in the mature paste.

Applications Expansive cements have been used in a wide variety of concrete structures. One of the more frequent uses in the United States has been in parking structures, to prevent water leaks that can cause damage to cars. The largest such structure is the parking building at O'Hare International Airport in Chicago, which used 90,000 m^3 (120,000 yd^3) of shrinkage-compensating concrete. Shrinkage control joints can be combined with thermal expansion control joints and other construction joints, and the elimination of shrinkage control joints is an attractive advantage in the laying of pavements. The largest paving job to date using this material is probably at the Love Field Airport at Dallas—Fort Worth, where more than 115,000 m^3 (150,000 yd^3) of shrinkage-compensating cement concrete was used in taxiways. A tunnel at the Cincinnati Airport (1992) consumed 58,000 m^3 (76,000 yd^3). Shrinkage-compensating cement can be used in concrete structures where watertightness is an important requirement, such as water storage tanks, swimming pools, and ice rinks. The use of expansive cement has also been found to be useful in tilt-up construction, where residual prestress can help the building elements to withstand stress imposed during lifting, as well as minimizing separations between elements. They can also be used to stabilize long-term dimensions of post-tensioned elements.

Self-Stressing Cements

We have seen that the mechanism of shrinkage compensation is the development of a *chemical prestress* sufficient to overcome tensile stresses induced by differential shrinkage strains. If greater amounts of prestress were developed, the residual compressive stress, after shrinkage is allowed for, could be used in design just as is done in mechanical prestressing. The origin of the prestress is of no concern provided that the required magnitude can be guaranteed. This is more difficult when the prestress is generated internally by a self-stressing cement than when it is applied externally by mechanical means. A self-stressing cement has high potential expansion, and the factors affecting expansion become even more critical. Close attention must be paid to the amount and positioning of restraining steel, mix design, and curing conditions for successful use of self-stressing cements. Further, the level of prestress that can be realistically attained will be relatively low, less than 3.5 MPa (500 lb/in.^{2}). Chemical prestressing has been used in concrete pressure pipe, water tanks, tunnel linings, and precast building elements.

Calcium Sulfoaluminate Cements

In the production of calcium sulfoaluminate cement, calcium sulfate is added to the raw mix so that $C_4A_3\overline{S}$ is formed in the rotary kiln. This is the same compound that is

present in Type E-1(K), but the quantities are greater here. Calcium sulfate ($C\overline{S}$, insoluble anhydrite) is also formed, or is added during grinding. A typical composition is given in Table 3.7, from which it can be seen that C_2S rather than C_3S is the calcium silicate formed because C_3S and $C_4A_3\overline{S}$ cannot coexist within the kiln. Free lime is absent in these formulations. Under low lime conditions during hydration, ettringite forms shorter and thicker needles, which reduce expansion and enhance strength.

Properties and Composition Strength development is initially similar to fluoroaluminate cement (see Figure 3.13) and strengths in excess of 7 MPa (1000 lb/in.^{2}) can be attained within 3 h of set. The formation of ettringite will be accompanied by a high rate of heat evolution, but if this early heat can be dissipated, the subsequent hydration of C_2S will not cause additional temperature rise. The presence of C_2S will also improve the durability of the cement paste because of the lower content of calcium hydroxide in the mix, but the long-term sulfate resistance of the cement may not be high. Creep and drying shrinkage are stated to be lower than for Type III cement concrete.

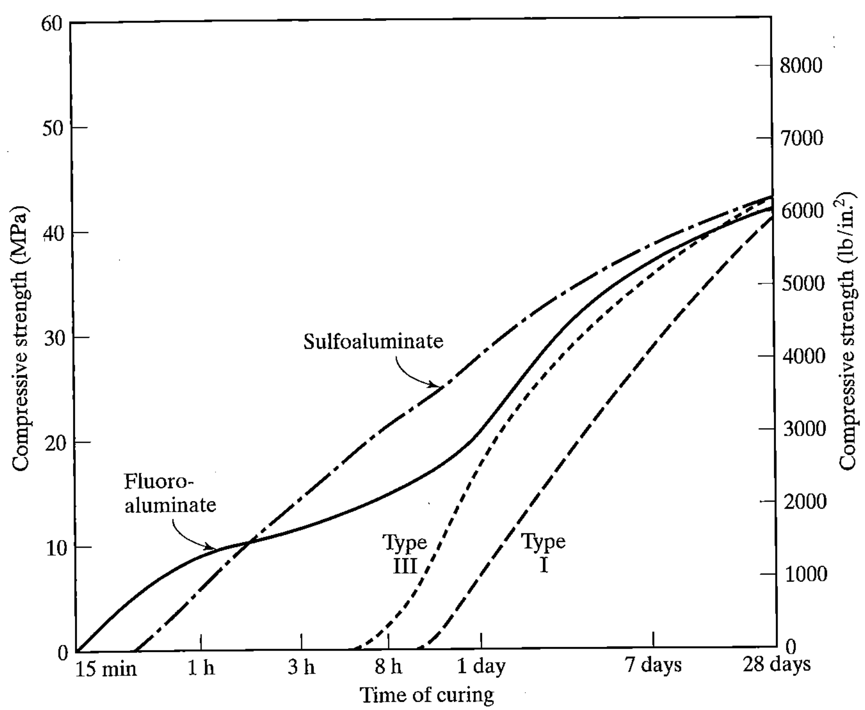


FIGURE 3.13

Strength development of concretes made with different rapid hardening cements. [Adapted from W. Perenchio in *New Materials in Concrete Construction*, University of Illinois at Chicago, p. 12-IV (1972).]

TABLE 3.7 Typical Compositions of Rapid-Setting Cements (%)

Cement Compounds	*Sulfoaluminate Cement*	*Fluoroaluminate Cement*	*Type I Portland (for comparison)*
C_3S	–	60	55
C_2S	30	5	19
C_3A	–	–	8
C_4AF	5	8	10
$C_4A_3\overline{S}$	55	–	–
$C_{11}A_7.CaF_2$	–	20	–
Total SO_3	10	10	5

Calcium Fluoroaluminate Cement

This cement is a modified portland cement (see Table 3.7) in which C_3A is replaced by a new compound, calcium fluoroaluminate, $C_{11}A_7 \cdot CaF_2$[1]. The cement can be produced directly in a rotary kiln, or the fluoroaluminate component can be blended with a Type I clinker. $C_{11}A_7 \cdot CaF_2$ is even more reactive with water than is C_3A and, in fact, hydrates so rapidly that flash setting will always occur unless sulfate ions are present. The reaction with sulfate ions to form ettringite is also very vigorous, so much so that gypsum will not successfully retard setting because it cannot dissolve fast enough to maintain an adequate supply of sulfate ions. The use of a more soluble salt, such as calcium sulfate hemihydrate (plaster) or sodium sulfate, is needed to avoid flash setting. The time of setting (handling time) can be controlled from about 2 min to 40 min using a soluble sulfate (see Figure 3.14) and further extended with an organic retarder such as citric acid. Hence, the alternative name *regulated-set cement*. The subsequent supply of sulfate ions required to form additional ettringite for strength development is maintained by slightly soluble anhydrite ($C\overline{S}$).

Strength development occurs very rapidly after setting occurs, and strengths of 7 MPa (1000 lb/in.^{2}) can be attained within 1 h (Figure 3.13). The initial strength rise is due to ettringite forming from $C_{11}A_7 \cdot CaF_2$. Once this reaction slows down, strength development also slows down, until C_3S begins to hydrate in the usual way.

Properties The level of the early-strength plateau depends on the amount of fluoroaluminate used, which can be controlled either by the cement content of the concrete or by the fluoroaluminate content of the cement. Strengths in excess of 20 MPa (3000 lb/in.^{2}) after 1 h have been achieved using cements with 50% by weight of $C_{11}A_7 \cdot CaF_2$. High early strength is generally accompanied by a relatively short handling time. If the handling time is extended by, for example, the use of citric acid, it will take longer to reach a given strength. A balance between an adequate handling time and a minimum rate of strength development may impose definite limitations on the cement composition.

The reaction of calcium fluoroaluminate is accompanied by the evolution of a large amount of heat, which exceeds that of a typical Type III cement. Even a small pat

[1] This is one of the few cement compounds that has a hybrid formula: $C_{11}A_7$ is cement notation, but CaF_2 is in conventional chemical symbols. It is not possible to express calcium fluoride by an oxide formulation.

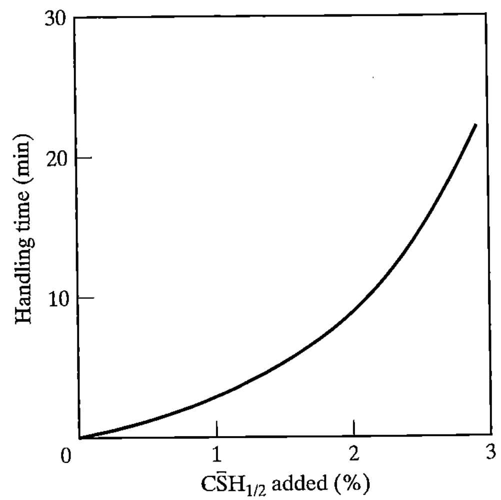


FIGURE 3.14

Effect of calcium sulfate hemihydrate on the handling time of calcium fluoroaluminate cement. [Adapted from *New Materials in Concrete Construction*, University of Illinois at Chicago, p. 10-IV (1972).]

of regulated-set cement paste quickly becomes warm to the touch. This high rate of heat evolution can help maintain an adequate concrete temperature in winter; if the concrete temperature falls below 4°C, rapid strength development does not occur. Most other concrete properties are much the same as those for ordinary portland cement concrete; but the high aluminate content means that a low sulfate resistance is to be expected, and this has been confirmed.

Applications One of the first applications visualized for regulated-set cement was in lightweight insulation for roof decks using expanded vermiculate aggregate. This type of concrete would match the rapid set of gypsum plaster, which is widely used, thereby permitting rapid placement and have the advantage of better water resistance. But the very rapid strength gain of the cement suggests many other applications in which the properties of a portland cement are desired: pavement and bridge-deck repair, precasting operations, shotcreting, and slipforming.

Miscellaneous Cements

Rapid Setting and Hardening Cements There is often a need for the development of significant strength within a few hours. While this can be achieved in many cases using low *w/c* ratios elevated curing temperatures, and accelerating admixtures, special cements also are produced for this purpose. Some are simply a Type III cement ground to a high fineness (700–900 m^2/kg), or ground with an accelerating admixture, or both. Early formation of ettringite is also used to provide high early strength as discussed in the two previous sections. Mixtures of Type I portland cement with gypsum are common. Magnesium phosphate cements also are sold for this purpose.

White Cement This is a popular cement with architects because it expands the opportunities for creating aesthetic effects. It is especially suitable for exposed aggregate finishes and for making colored cements with pigment additions. The white color is

achieved by eliminating iron from the cement; it is thus a Type I or Type III cement with a high C_3A content and no C_4AF. Iron-free clay (kaolinite or china clay) must be used, and bauxite (aluminum oxide) is often needed to achieve the required alumina content. Special ball mills must be used to prevent iron contamination during grinding. The higher cost of raw materials and changes in manufacturing procedures make white cement expensive.

Masonry Cement Mortars used for laying brick and block have special requirements for workability, plasticity, and water retention. To ensure uniform characteristics, masonry cements conforming to ASTM C 91 or CSA A8 are generally specified. They are essentially a Type I portland cement mixed with finely divided materials such as hydrated lime, talc, clay, fly ash, slag, or ground limestone, together with an air-entraining agent. These additives provide the "fattiness" and cohesiveness of a good mortar, improve its adherence to the brick or block, prevent bleeding, and minimize water loss due to absorption. The air-entraining agent also provides frost resistance.

Oil-Well Cements Cements used for sealing oil wells must be slow setting even at high temperatures and pressures, and stable under highly corrosive conditions. The American Petroleum Institute has written specifications for nine different classes of cements applicable at certain well depths. Generally, low C_3A portland cements are used together with admixtures that retard setting.

Cements with no C_3A at all are used as depths increase, and the improved sulfate resistance of these cements is an advantage since groundwaters frequently contain sulfate. In the deepest wells, where temperatures may exceed 260°C (500°F), cements based largely on C_2S are used to prolong setting times at these temperatures. Oil-well cements are pumped as slurries with high water contents. Finely divided materials, such as bentonite or other clays, and organic thickening agents, are used to prevent water loss and segregation while maintaining pumpability. The important parameter is not setting time as measured by penetration tests, but *thickening time*, which determines how long a slurry can be pumped.

Natural Cement Early cement production in the United States used "cement rock," which is an argillaceous (clayey) limestone with a composition similar to the raw mix from which portland cement is made. The cement rock is burned at lower temperatures than are now used for portland cement; thus, very little C_3S is formed, and the cement contains mostly C_2S. The cement is therefore slow hardening and somewhat variable in performance, since no compositional blending was done. Natural cements are not used today.

3.4 NON-PORTLAND INORGANIC CEMENTS

There are a number of other cements that are used to some extent in building and construction. Although they are not often encountered, a brief discussion of their composition and performance is warranted.

Calcium Aluminate Cement

Calcium aluminate cement (CAC), which is also known as high alumina cement (HAC), deserves consideration because it is widely available and has been used in structural applications in some countries. CAC was originally developed as a sulfate-resistant cement, but now is used primarily in refractory concretes.

Composition and Chemistry Unlike portland cement, the manufacture of iron-rich CAC usually involves complete fusion of the raw materials in a kiln or furnace. However, iron-poor CAC is produced by a sintering process in which the pelletized feed is passed through a specially designed rotary kiln. The major chemical component is monocalcium aluminate (CA). Other compounds that are typically present are listed in Table 3.8, but their proportions may vary considerably from one cement to another. The iron content in particular may vary widely, from 15% to less than 2%. After an initial period of quiescence, CA hydrates quite rapidly. Thus, although CAC has a setting time comparable to portland cements, its early strength gain is very rapid. Within 24 h of mixing, concrete strengths can attain values exceeding the seven-day strengths of portland cements (see Figure 3.15), which represents about three-fourths of the ultimate potential strength. Furthermore, strength gain at low ambient temperatures is much more satisfactory than it is with portland cement, due in part to the high heat of hydration.

Conversion of CAC CA hydrates to form calcium aluminate hydrates, the compositions of which depend on the temperature of hydration.

$$CA + H \begin{cases} \xrightarrow{<10^\circ C} CAH_{10} \\ \xrightarrow[10\text{–}30^\circ C]{} C_2AH_8 + AH_3 \\ \xrightarrow[>30^\circ C]{} C_3AH_6 + 2AH_3 \end{cases} \quad (3.4)$$

Under normal ambient conditions, a mixture of CAH_{10} and C_2AH_8 can be expected to form. Both of these hydrates are metastable with respect to C_3AH_6; if the temperature rises above 30°C (86°F), they will transform to the latter. This transformation is called *conversion*, and it leads to loss in strength because of an accompanying decrease in solid volume. This results in an increase in porosity, which can be measured experimentally,

TABLE 3.8 Typical Compositions of Calcium Aluminate Cements (%)

Constituents	*Compound*	*Iron-rich*	*Iron-poor*
Major	CA	60	70
Intermediate	C_2S	10	–
	C_2AS (gehlinite)	5–20	5
	CA_2	–	20
Minor	$C_{12}A_7$	10–25	< 5
	FeO (wustite)		
	C_4AF		
	Pleochroite		

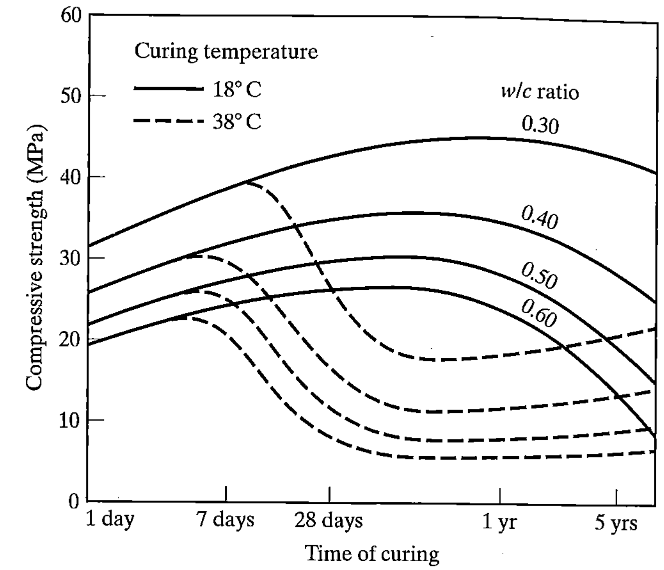


FIGURE 3.15

Effect of curing temperature on strength development of calcium aluminate cement concretes.

and a disruption of the original paste microstructure. The increase in porosity is the most important factor as it not only strongly affects strength, but also accelerates corrosion of reinforcing steel. Although exposure to hot moist conditions is conducive to conversion, it will proceed slowly even at temperatures around 20°C (68°F). This is because the rapid evolution of heat during early hydration raises the internal temperature of the concrete to well above 30°C (86°F) and initiates conversion. Once C_3AH_6 has nucleated, it can continue to form at lower temperatures.

Strength loss due to conversion can be controlled in two ways. If the concrete is moist-cured at 80°C (176°F) or higher, C_3AH_6 will form directly to form a strong, stable microstructure. If the *w/c* ratio is kept below 0.4, strength loss below 25°C (77°F) due to conversion will be minimized and any reduction will be temporary (see Figure 3.15). This is because residual unreacted cement can begin to hydrate when the porosity is increased, thereby reducing it again. Although we understand the consequences of conversion and how to control it, the use of CAC remains banned in most countries. It is used primarily for concrete or mortar that will be exposed to high temperatures, or for castable refractories (firebrick). It is also useful for rapid repair work because of its high rate of strength gain, particularly at low temperatures. Such repairs may not be durable, however, due to subsequent conversion.

Gypsum Plaster

Plaster products have a widespread use in the building industry as a surface finish on interior walls and in the production of drywall products for interior lining and partitioning where structural requirements are low. Plasters are calcium sulfate cements that rely on the formation of calcium sulfate dihydrate (gypsum) for strength development. Plaster is β-calcium sulfate hemihydrate ($CaSO_4 \cdot \frac{1}{2}H_2O$) formed by controlled

heating of natural gypsum. On mixing with water, gypsum is reconstituted by rehydration. If calcination temperatures are too high, anhydrite is formed, which is much less reactive. Dehydration occurs slowly at temperatures below 150°C (300°F),

$$\underset{\text{gypsum}}{C\bar{S}H_2} \underset{H_2O}{\overset{150^\circ C}{\rightleftarrows}} \underset{\substack{\beta\text{-hemihydrate}\\ \text{or plaster}}}{C\bar{S}H_{1/2}} \xrightarrow{300^\circ C} \underset{\text{anhydrite}}{C\bar{S}} \tag{3.5}$$

but this temperature is chosen to ensure complete decomposition.

Gypsum plasters have the advantage of being quick-setting materials that develop strength rapidly. Set time can be controlled through the manufacturing process or by use of special admixtures. The major disadvantage of gypsum plaster as a structural material is its lack of water resistance. Gypsum is quite soluble in water, which makes its use impractical where contact with water is to be expected. It should be noted that leachates from plaster are rich in sulfate ions and the proximity of plaster and portland cement concretes when leaching occurs can lead to sulfate attack.

Other Cements

There are a number of other non-portland, inorganic cements, such as magnesium oxychloride cement (Sorel cement) and phosphate-bonded cements (used as a rapid patching material for highways). A discussion of these cements, however, is beyond the scope of this book.

3.5 SPECIFICATIONS AND TESTS OF PORTLAND CEMENT

The production of portland cement requires strict quality control, which has made necessary the establishment of specifications for both the chemical and physical requirements of cement. This, in turn, has involved the development of a number of "standard" tests that can be carried out relatively easily and rapidly to ensure that the cement is indeed of the desired quality. It should be noted that different countries have established different tests and specifications, and that even within a country these are subject to frequent revision. Unfortunately, it is usually not possible to compare directly the results obtained by different test methods, though correlations can often be established between them, because the tests do not measure any *fundamental* properties of the cement. Rather, they simply provide information as to whether the cement meets certain (often arbitrary) standards.

Since different test methods for the same property often give quite different results, the test methods used must also be indicated when describing the properties of cement. In North America, the most common tests are those developed by ASTM, and it is primarily these tests that will be outlined later, though other test methods will be described if they differ substantially from the ASTM procedures.

Chemical Requirements

The chemical requirements for portland cement given in ASTM C 150 are shown in Table 3.9. These specifications allow wide variations, because it has been found that

cements with quite different chemical compositions may have suitable physical behavior. The only limits placed on all cement types are the MgO and SO_3 levels and, optionally, alkalis.

Physical Requirements

The physical requirements for portland cement given in ASTM C 150 and shown in Table 3.10 are more significant. The tests for these (and a few other) requirements will be described subsequently.

TABLE 3.9 Chemical Requirements of ASTM C 150 Standard Specification for Portland Cement[g]

Cement Type	*I and IA*[f]	*II and IIA*	*III and IIIA*	*IV*	*V*	*Remarks*
Standard requirements						
Silicon dioxide (SiO_2), min., %	–	20.0	–	–	–	
Aluminum oxide (Al_2O_3), max., %	–	6.0	–	–	–	
Ferric oxide (Fe_2O_3), max., %	–	6.0	–	6.5	–	
Magnesium oxide (MgO), max., %	6.0	6.0	6.0	6.0	6.0	
Sulfur trioxide (SO_3), max., %						
When ($3CaO \cdot Al_2O_3$) is 8% or less	3.0	3.0	3.5	2.3	2.3	
When ($3CaO \cdot Al_2O_3$) more than 8%	3.5	N.A.[a]	4.5	N.A.	N.A.	
Loss on ignition, max., %	3.0	3.0	3.0	2.5	3.0	
Insoluble residue, max., %	0.75	0.75	0.75	0.75	0.75	
Tricalcium silicate (C_3S), max., %[b]	–	–	–	35[d]	–	
Dicalcium silicate (C_2S), min., %[b]	–	–	–	40[d]	–	
Tricalcium aluminate (C_3A), max., %[b]	–	8	15	7	5	
Tetracalcium aluminoferrite+ 2 (tricalcium aluminate)(C_4AF+ $2C_3A$) or solid solution (C_4AF+ C_2F) as applicable, max., %	–	–	–	–	20	
Optional requirements[c]						
Tricalcium aluminate (C_3A), max., %	–	–	8	–	–	For moderate sulfate resistance
Tricalcium aluminate (C_3A), max., %	–	–	5	–	–	For high sulfate resistance
Sum of tricalcium silicate and tricalcium aluminate, max., %	–	58[d]	–	–	–	For moderate heat of hydration
Alkalies (Na_2O + $0.685K_2O$), max., %	0.60[e]	0.60[e]	0.60[e]	0.60[e]	0.60[e]	Low-alkali cement

[a] N.A., not applicable.
[b] The expressing of chemical limitations by means of calculated compounds does not necessarily mean that the oxides are entirely present as such compounds, or that the compounds have the exact stoichiometry implied.
[c] These apply only when specifically requested.
[d] This limit applies when a limit on heat of hydration is not specified.
[e] This limit may be specified when the cement is to be used in concrete with aggregates that may be deleteriously reactive.
[f] Types IA, IIA, and IIIA are air-entraining cements.
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TABLE 3.10 Physical Requirements of ASTM C 150 Standard Specification for Portland Cement[g]

Cement Type	*I*	*IA*[f]	*II*	*IIA*	*III*	*IIIA*	*IV*	*V*
Standard requirements								
Air content of mortar,[a] vol. %								
Max.	12	22	12	22	12	22	12	12
Min.	–	16	–	16	–	16	–	–
Fineness, specific surface, m²/kg (alternative methods)								
Turbidimeter test, min.	160	160	160	160	–	–	160	160
Air permeability test, min.	280	280	280	280	–	–	280	280
Autoclave expansion, max., %	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
Strength, not less than the values shown for the ages indicated below								
Compressive strength, MPa (lb/in.²)								
1 day	–	–	–	–	12.0 (1740)	10.0 (1450)	–	–
3 days	12.0 (1740)	10.0 (1450)	10.0 (1450) 7.0[b] (1020)	8.0 (1160) 6.0[b] (870)	24.0 (3480)	19.0 (2760)	–	8.0 (1160)
7 days	19.0 (2760)	16.0 (2320)	17.0 (2470) 12.0[b] (1740)	14.0 (2030) 9.0[b] (1310)	–	–	7.0 (1020)	15.0 (2180)
28 days	–	–	–	–	–	–	17.0 (2470)	21.0 (3050)
Time of setting (alternative methods)								
Gillmore test								
Initial set, min., not less than	60	60	60	60	60	60	60	60
Final set, min, not more than	600	600	600	600	600	600	600	600
Vicat test								
Initial set, min., not less than	45	45	45	45	45	45	45	45
Final set, min, not more than	375	375	375	375	375	375	375	375
Optional requirements[c]								
False set, final penetration, min., %	50	50	50	50	50	50	50	50
Heat of hydration[d]								
7 days, max, kJ/kg (cal/g)	–	–	290 (70)	290 (70)	–	–	250 (60)	–
28 days, max, kJ/kg (cal/g)	–	–	–	–	–	–	290 (70)	–
Strength, not less than values shown								
Compressive strength, MPa (lb/in.²) 28 days	28.0 (4060)	22.0 (3190)	28.0 (4060) 22.0[b] (3190)	22.0 (3190) 18.0[b] (2610)	–	–	–	–
Sulfate expansion[e], 14 days, max., %	–	–	–	–	–	–	–	0.040

[a] Compliance with the requirements of this specification does not necessarily ensure that the desired air content will be obtained in concrete.
[b] When the optional heat of hydration or the chemical limit on ($C_3S + C_3A$) is specified.
[c] These apply only when specifically requested.
[d] When the heat of hydration is specified, ($C_3S + C_3A$) is not limited.
[e] When sulfate expansion is specified, C_3A and ($C_4AF + 2C_3A$) are not limited.
[f] Types IA, IIA, and IIIA are air-entraining cements.
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Fineness

As has been pointed out, the last step in the production of portland cement is the grinding of clinker and gypsum. The fineness to which the cement is ground can have a considerable effect on the behavior of the cement during hydration:

1. Of greatest importance, the rate of hydration increases with increasing fineness. This leads to both a higher rate of strength gain (see Section 13.7) and a higher rate of evolution of heat.
2. Increasing fineness tends to decrease the amount of bleeding, but at high fineness the amount of water required for workability for non-air-entrained concrete is increased, which results in increased drying shrinkage.
3. A high cement fineness reduces the durability of concrete to freeze–thaw cycles.
4. An increased fineness requires a greater amount of gypsum for proper set control, owing to the increased availability of C_3A for reaction.

There are really two things that are of interest: the specific surface of the particles (i.e., the summation of the surface areas of all of the particles in 1 g or 1 kg of cement) and the particle-size distribution. However, it is general practice now to describe fineness by a single parameter, the specific surface area. Although cements of quite different particle-size *distributions* might have the same specific surface area, this is still considered to be the most useful measure of cement fineness. And although it is possible to measure particle-size distributions, there is still no agreement on what would constitute a "best" grading curve for cement. It must be noted that for the irregularly shaped cement particles, no two methods of measuring specific surface will give the same results. This is partly because the different methods tend to measure somewhat different properties, and partly because of the assumptions required to develop mathematical relationships. Therefore, wherever specific surface is given, the method used to measure it must also be given. The real use of any given method is to allow a comparison between different cements, and thus provide a tool for both quality control and research. Two methods of determining the fineness of cement are recognized by ASTM: the turbidimeter test and the air permeability test.

Wagner Turbidimeter The Wagner turbidimeter (ASTM C 115) method of measuring specific surface involves preparing a suspension of cement in kerosene in a tall glass container. Parallel rays of light are then passed through the container onto a photoelectric cell, and the cross-sectional area of the particles intersecting the beam can be determined by measuring the light intensity. This test is based on *Stokes' law*, which states that when a small sphere falls through a viscous medium under the action of gravity, it ultimately acquires a constant velocity

$$V = \frac{2ga^2(d_1 - d_2)}{9\eta} \tag{3.6}$$

where g is the acceleration due to gravity, a the radius of the sphere, d_1 the density of the sphere, d_2 the density of the viscous medium, and η the viscosity. Readings are taken at given times and heights below the surface of the kerosene. From these data, a

specific surface area and a particle-size distribution can be obtained. In these calculations, the assumptions are made that (1) the particles are spherical, (2) the flow is truly "viscous," and (3) particles in the range of 0 to 7.5 μm have a uniform size of 3.75 μm.

Although the turbidimeter values do not correlate well with the strength of cement, this method does provide a fairly rapid and reproducible measure of the relative fineness of cement. Automated instruments are available that use the sedimentation method to determine particle-size analysis rapidly, but they are not recognized in ASTM tests.

Blaine Air-Permeability Apparatus The air-permeability method of determining the specific surface (ASTM C 204) is based on the relationship between the surface area of the particles in a porous bed and the rate of fluid flow through the bed. The basic equation developed by Carman is

$$S = \frac{14}{D(1-\epsilon)} \frac{\sqrt{\epsilon^3 A i}}{\nu Q}, \tag{3.7}$$

where S is the specific surface (cm^2/g), D the powder density, ϵ the porosity of the bed, A the cross-sectional area, i the hydraulic gradient, ν the kinematic viscosity, and Q the rate of flow. In the Blaine method (ASTM C 204), rather than passing air through the bed at a constant rate, a given volume of air is passed through a bed of standard porosity at a steady diminishing rate, and the time (t) required is measured. The specific surface (S) is then calculated from the relationship

$$S = K\sqrt{t} \tag{3.8}$$

where K is a constant. In practice, K is not determined directly; rather, the sample is compared to a standard sample of known surface area issued by the U.S. National Institute of Standards and Technology.

In summary, both the Wagner and Blaine methods provide an acceptable way of measuring surface areas. The Blaine value is generally about 1.8 times the Wagner value, probably because of the different theories involved. The Blaine method is more commonly used, but in cases of dispute the Wagner method is deemed to govern.

Tests on Cement Paste

Normal Consistency Two of the common physical requirements for cement paste, time of setting and soundness, depend on the water content of the neat cement paste. Therefore, it is necessary to define a water content at which to do these tests. This is defined in terms of the *normal consistency*, measured according to ASTM C 187. This test is based on the depth of penetration of a 10-mm-diameter needle under a load of 300 g into fresh cement paste, using the Vicat apparatus shown in Figure 3.16. A paste is said to have normal consistency when the plunger penetrates 10 ± 1 mm below the original surface in 30 s. The amount of water required is expressed as a percentage by weight of the dry cement, the usual range being about 24 to 33%. This test is a measure of the plasticity of the cement paste, although it does not correlate particularly with the

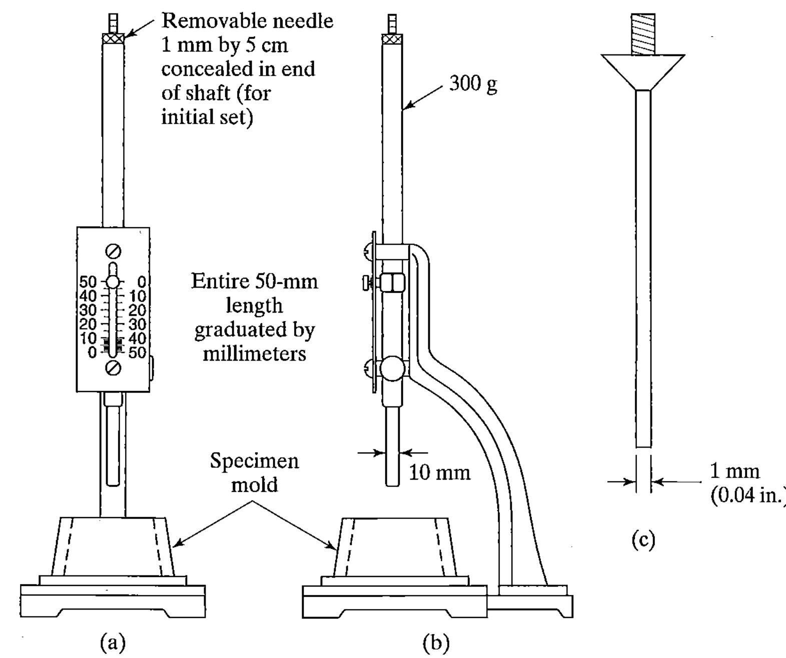


FIGURE 3.16

Vicat apparatus: (a, b) front and side view, respectively, of apparatus setup for consistency tests; (c) enlarged view of needle used to determine final set. (Adapted from ASTM C 187 and C 191. Copyright ASTM INTERNATIONAL. Reprinted with permission.

quality of the cement. It is very sensitive to the conditions under which it is carried out, particularly temperature and the way the cement is compacted into the mold.

Time of Setting The chemical reactions that occur during the setting and hardening of cement will be discussed in Chapter 4. We are here concerned only with the measurement of the setting time. The setting times defined next are of no fundamental significance; they merely define two arbitrary points in the general relationship between the time after the addition of water and strength gain. Two setting times are defined:

1. *Initial set*, which indicates that the paste is beginning to stiffen considerably and can no longer be molded;
2. *Final set*, which indicates that the cement has hardened to the point at which it can sustain some load.

These tests are used primarily for quality control. Generally, initial set occurs in 2 to 4 h, and final set in 5 to 8 h. Concretes, which generally have higher water contents, tend to set more slowly. There are two commonly accepted test methods for determining the setting time of cement paste. They are both carried out on pastes of normal consistency.

Time of setting by the Vicat needle (ASTM C 191). In this test, the Vicat apparatus illustrated in Figure 3.16 is used, except that the 1-mm-diameter needle is used for penetration (Figure 3.16c). The initial setting time is defined as the time at which the needle penetrates 25 mm into the cement paste. The final set occurs when the needle does not sink visibly into the paste.

Time of setting by Gillmore needle (ASTM C 266). This test is less commonly used than the Vicat test and gives different results. The apparatus is shown in Figure 3.17. The cement is considered to have acquired initial set when the initial Gillmore needle, weighing 113.4 g ($\frac{1}{4}$ lb) and with a diameter of 2.12 mm ($\frac{1}{12}$ in.), fails to penetrate. Final set is reached when the final Gillmore needle, weighing 453.6 g (1 lb) and with a 1.06 mm ($\frac{1}{24}$ in.) diameter, fails to penetrate.

Gillmore times tend to be longer than Vicat times. Recent research has shown that setting times determined by the Vicat needle correspond closely to significant changes in the fluid properties of the paste. A modified version of the Vicat test can be used to determine the setting time of mortars (ASTM C 807).

Early Stiffening The problem of early stiffening of the cement paste may be manifested by two different phenomena (which are discussed in more detail in Chapter 9):

1. *False set.* This refers to the rapid development of rigidity in cement paste, without the evolution of much heat. The rigidity can be overcome and plasticity regained by further mixing, but with no addition of water.

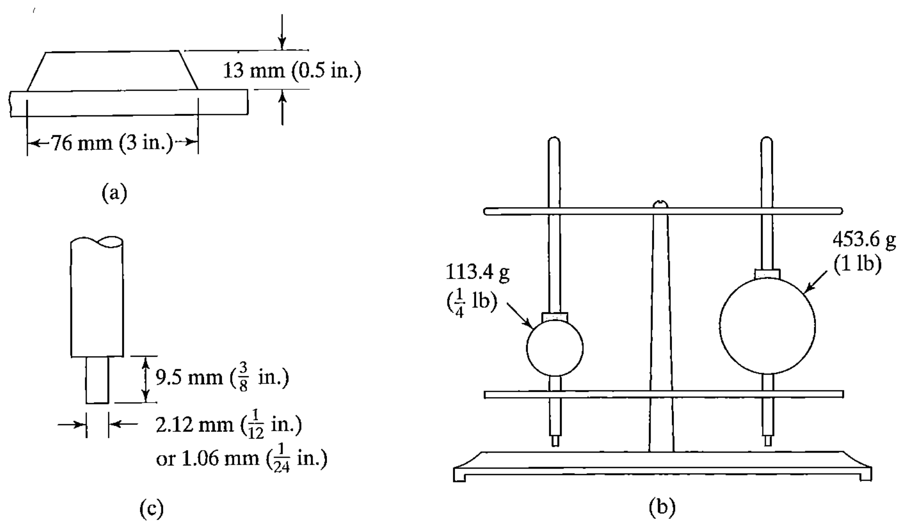


FIGURE 3.17

Gillmore apparatus: (a) test pat; (b) weighted needles; and (c) detail of needle tips. (Adapted from ASTM C 266. Copyright ASTM INTERNATIONAL. Reprinted with permission.

2. *Flash set (or quick set).* This refers to the rapid development of rigidity in cement paste, *with* the evolution of considerable heat. The rigidity cannot be overcome, and the plasticity cannot be regained.

The test for early stiffening is described in ASTM C 451, using the Vicat apparatus. A paste is prepared with enough water so that the initial penetration 20 s after the completion of mixing is 32 ± 4 mm. The final penetration is determined 5 min after the completion of mixing. The results are reported as (final penetration/initial penetration) × 100%. This test is used to determine whether the cement complies with the requirements of Table 3.10. It also provides information as to whether the cement is likely to exhibit early stiffening. Although false set may make handling and placing the cement difficult, it is not deleterious to the concrete quality. The problem of false set will not occur with transit mixing, where the concrete is mixed for a long time before placing, or where the concrete is remixed prior to placement, as with pumping. The consequences of flash set are more severe and will usually cause the cement to fail to meet the time of set requirements in Table 3.10.

Unsoundness Unsoundness in cement paste results from excessive volume change after setting. If there is any appreciable expansion, however slow, cracking and failure of the concrete will result. Unsoundness in cement is caused by the slow hydration of MgO or free lime, and by the reactions of gypsum with C_3A (see Section 3.1). These expansive reactions take place very slowly, and so unsoundness will only appear after many months, or even years. Therefore, it is necessary to use some form of accelerated test, so that tendencies toward unsoundness can be detected as a quality control measure. Although a number of soundness tests have been developed over the years, only one is in common use.

The autoclave expansion test (ASTM C 151) will detect unsoundness due to both excess CaO and excess MgO, and is the test designated in ASTM C 150. A cement of normal consistency is molded and cured normally for 24 h. The specimens are then removed from the molds, measured, and placed in an autoclave. The temperature is raised so that the steam pressure in the autoclave reaches 2 MPa (295 lb/in.^{2}) in 45 to 75 min; this pressure is maintained for 3 h, and the autoclave cooled so that the pressure is relieved in $1\frac{1}{2}$ h. The specimen is then cooled in water, to 23°C in 15 min. Its length is again measured after a further 15 min; the expansion must be less than 0.80% if the cement is to be acceptable.

These accelerated tests clearly do not simulate service conditions, and the amount of expansion may be affected by a number of factors, such as the fineness of the cement and the C_3A content. Thus, these tests can only serve as a guide and cannot provide an absolute indication of a tendency toward unsoundness for a given cement. However, they provide a valuable quality control measure, since changes in the expansion tests would indicate some anomaly in the cement manufacturing process.

Heat of Hydration The heat of hydration varies greatly with the cement composition, with C_3A and C_3S being primarily responsible for high heat evolution [see Eqs. (3.1a) and (3.1b) and Figure 3.6]. Fineness of grinding is important with regard to the *rate* of heat evolution. Finely ground cements increase the hydration rate, but the total heat of

hydration at very long ages is not particularly affected. The heat of hydration may be defined as the amount of heat evolved during the setting and hardening of portland cement at a given temperature measured in J/g of unhydrated cement. This is most commonly done by the method specified in ASTM C 186, the heat of solution method. Basically, the heat of solution of dry cement is compared to the heats of solution of separate portions of the cement that have been partially hydrated for 7 and 28 days. The heat of hydration is then the difference between the heats of solution of the dry and partially hydrated cements for the appropriate hydration period. As may be seen from Table 3.10, only Type II and Type IV cements have heat-of-hydration requirements.

Tests on Mortar

While some of the specifications for cement require tests done on neat pastes, others require that the tests be carried out on mortars. Since such tests clearly depend on the type of sand used, the mortar tests defined by ASTM all use a graded standard sand. This is a natural silica sand from Ottawa, Illinois, with a specific grading between 150 μm (No. 100) and 1.18 mm (No. 16) sieves as defined in ASTM C 778. Sands from different sources may give quite different results even though they meet this grading requirement.

Mortar Flow As with cement pastes, many of the properties of mortars also depend on the consistency, or *w/c* ratio. While some specifications for mortar tests are written in terms of a fixed *w/c* ratio, others are written in terms of consistency. The consistency of mortars is expressed as mortar flow, determined according to the procedures of ASTM C 1437. Mortar is prepared with a ratio of 2.75 parts Ottawa sand to 1 part cement (by weight). It is compacted in a mold in the form of a truncated cone 2 in. (50.8 mm) deep, with a bottom diameter of 4 in. (102 mm) and a top diameter of 2 $^3/_4$ in. (69.8 mm). The cone is placed on a flow table, that is, a table whose top can be raised and dropped through a height of 0.5 in. (12.7 mm) by means of a rotating cam. The mold is removed from the mortar, and the table is dropped 25 times in 15 s. The *flow* is then the resulting increase in the average base diameter of the mortar mass, measured as a percentage of the original diameter.

Strength Tests. Since cement is used primarily as a structural material, its strength properties are of prime importance. Therefore, a number of strength tests have been developed to try to answer two questions:

1. What will be the strength of concrete made with a particular cement?
2. How do different cements compare with one another?

Unfortunately, strength is not a very easy property to define, as will be discussed in more detail in Chapter 13. The factors that can influence the measurement of strength include the *w/c* ratio, cement/sand ratio, type and grading of sand, manner of mixing and molding specimens, curing conditions, size of specimen, shape of specimen, moisture content at time of test, loading conditions, and age. In view of these many variables, it should be clear that any strength test must follow the specified testing procedure

very closely. Before we examine the test methods themselves, two of these variables must be discussed further: sand and age.

Ever since the testing of cement began, the question has been raised as to whether strength tests should be carried out on neat pastes or on mortars. Although it would appear to be most logical to carry out the test on a neat paste, since the use of sand introduces a number of extra variables, such tests are rarely used today. The problem with neat cement paste is that it is difficult to handle and test, and thus more variability is introduced into the results. It has also been found that cements that appear to be the same when tested neat may behave quite differently when used in mortar, and so mortar tests provide a more reliable indication of cement quality. Tests on neat cement pastes are now used only for research purposes. However, having decided to use mortar, the sand must be very carefully specified, as is done in ASTM standards.

Since the strength of hydrated cement increases with time, it is also necessary to specify the age at which tests should occur. Normally, minimum strengths are specified for 3, 7, and 28 days. In addition, high-early-strength cement (Type III) has a 1-day requirement, and the low-heat cements (Type IV), which hydrate slowly, may sometimes have a 90-day requirement as well. The requirements of ASTM C 150 are given in Table 3.10. The point to remember is that the shape of the strength–time relationship is important, not simply the strength at the single given time. Strength can be measured in compression, tension, or flexure.

Compressive strength. This is by far the most common measure of strength required by cement specifications. The test prescribed in ASTM C 109 uses as the test specimen a 50mm (2-in.) mortar cube. The sand/cement ratio is 2.75:1, using the standard Ottawa sand. The *w/c* ratio is 0.485 for all portland cements, and 0.460 for air-entraining portland cements. (However, some other specifications express the water requirement in terms of mortar flow.) The mortar is mixed according to a certain schedule, compacted into the molds, and placed in a moist storage room for 24 h. The specimens are then removed from the molds and stored in saturated lime water at 23°C (73.4°F) until tested. They are tested wet, using a machine with a spherically seated block. The loading rate is such that the specimen will fail in 20 to 80 s.

As an alternative to this test, compressive strength determinations may also be made on portions of test prisms broken in flexure (see next subsection), as described in ASTM C 349 (modified cube method). The two broken portions of the flexural specimen (originally 40 × 40 × 160 mm) are both tested. The test is carried out by loading each portion through bearing plates in the form of 40 × 50.8 mm rectangles. The strength measured on these modified cubes is not directly comparable to the standard cube strength. The compressive strength S_c of the modified cubes is calculated as

$$S_c = 0.62P \quad \text{(kPa)} \tag{3.9a}$$

$$S_c = 0.40P \quad \text{(lb/in.}^2\text{)} \tag{3.9b}$$

where P is the maximum load in pounds or newtons.

Flexural strength. The flexural test (which is really a measure of the tensile strength in bending) has gained considerable popularity lately, particularly in Europe. This is partly because the same specimen may then be used for compressive strength

determinations using the modified cube method just described. In ASTM C 348, which is somewhat different from the European method, specimens are cast in the form of 40 × 40 × 160 mm prisms. The proportioning and mixing of the mortar are the same as for the compression test. The specimens are then tested in flexure in center-point loading, and the flexural strength calculated.

It must be remembered, however, that the strengths of mortar as determined by these tests cannot be related directly to the strengths of concrete made with the same cements. Thus, the strength tests on mortars serve primarily as quality-control tests. The strength of concrete can only be determined from tests done on the concrete itself. In addition, the tests and specifications described here were developed for cements to be used in normal-strength concretes. They do not guarantee optimum performance for high-strength concretes, as described in Chapter 19.

Air Content of Mortar The purpose of the test for the air content of mortar (ASTM C 185) is to determine the air-entraining potential of a given cement sample. For ordinary cement, the purpose is to ensure that the cement will not entrain undesired air; for air-entraining cements, the purpose is to ensure that the additions are present in the correct quantity. Since the air content of concrete depends on a number of factors, the results of tests on mortar cannot be closely correlated to the air content of concrete made with a particular cement. However, ASTM C 150 does specify maximum and minimum air contents of the mortar.

In the test, mortar is made with cement and a standard sand, graded so that all the particles lie between the No. 16 and No. 30 sieves (ASTM C 778). The water content is chosen to give a flow of 87.5 ± 7%. The mortar is then compacted lightly into a 400-ml cup and the weight of mortar determined. The volumetric air content is calculated from the measured density and that of the air-free mixture, determined from the mixture proportions and the separate densities of the constituents. However, difficulty may be encountered in carrying out this test, in the form of apparent high air contents that occur occasionally with certain lots of sand.

Sulfate Expansion There are no really suitable tests available for determining the resistance of a cement to sulfate attack, but ASTM C 452 can provide useful information and is required for Type V cements. This test measures the expansion of mortar bars made from a mixture of cement and gypsum such that the total SO_3 content is 7.0% by weight. Mortar specimens are prepared in the form of 25 × 25 × 285 mm (1 × 1 × 11 $\frac{1}{4}$ in.) bars, using graded standard sand, a sand/(cement + gypsum) ratio of 2.75 and a water/(cement + gypsum) ratio of 0.485. After casting, the specimens are stored in water at 23°C (73.4°F), and the lengths determined at different times. The expansion is then a measure of the sulfate resistance of the cement and should not exceed 0.045% after 14 days.

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PROBLEMS

3.1. Calculate the compound composition for a portland cement having the following oxide analysis:
   a. C = 64.15%, S = 21.87%, A = 5.35%, F = 3.62%, $\bar{S}$ = 2.53%
   b. C = 64.15%, S = 21.37%, A = 5.35%, F = 3.62%, $\bar{S}$ = 2.53%
   c. C = 64.15%, S = 21.87%, A = 6.02%, F = 2.63%, $\bar{S}$ = 2.84%
   d. C = 63.54%, S = 23.09%, A = 3.61%, F = 6.38%, $\bar{S}$ = 2.29%

3.2. Calculate the heat of hydration after three days for a portland cement with the following compound composition:
   a. C_3S = 55%, C_2S = 24%, C_3A = 10%, C_4AF = 9%
   b. C_3S = 27%, C_2S = 51%, C_3A = 7%, C_4AF = 13%

3.3. Assign each of the following cements to one of the ASTM Types I to V:
   a. C_3S = 55%, C_2S = 21%, C_3A = 11%, C_4AF = 8%, fineness 480 m^2/kg
   b. C_3S = 55%, C_2S = 22%, C_3A = 10%, C_4AF = 8%, fineness 380 m^2/kg
   c. C_3S = 44%, C_2S = 34%, C_3A = 4%, C_4AF = 13%, fineness 370 m^2/kg
   d. C_3S = 30%, C_2S = 45%, C_3A = 6%, C_4AF = 12%, fineness 320 m^2/kg

3.4. Why is gypsum interground with portland cement clinker?

3.5. Why must particular attention be paid to curing expansive cements?

3.6. Why must expansive cement concretes be restrained during hardening?

3.7. What cements would be appropriate for pavement repair under winter conditions?

3.8. Why is high-alumina cement not allowed to be used structurally?

3.9. Why is it important to follow standard tests exactly when testing portland cement?

3.10. Can the behavior of concrete be accurately predicted from the results of the standard tests used to evaluate the performance of portland cement?

3.11. Is C_3A a useful component of portland cement?

3.12. What role does ettringite play in cement technology?

CHAPTER 4

Hydration of Portland Cement

4.1 CHEMISTRY OF HYDRATION

The setting and hardening of concrete are the result of chemical and physical processes that take place between cement and water. An adequate understanding of the chemistry of hydration is necessary for a full appreciation of the properties of cements and concretes and is discussed in detail in this chapter. The underlying chemistry has already been applied in Chapter 3 and will be referred to again in later chapters, particularly those dealing with admixtures and durability.

Hydration of Pure Cement Compounds

The chemical reactions describing the hydration of the cement compounds have been worked out through the study of the hydration of pure cement compounds. It is assumed that the hydration of each compound takes place independently of the others that are present in portland cement. This assumption is not completely valid, since interactions between hydrating compounds can have important consequences, but in most cases it provides a reasonable approximation to understanding how cement hydrates.

Calcium Silicates The hydration reactions of the two calcium silicates [Eqs. (4.1) and (4.2)] are stoichiometrically very similar, differing only in the amount of calcium hydroxide formed:

$$2C_3S + 11H \rightarrow C_3S_2H_8 + 3CH \qquad (4.1)$$

tricalcium silicate	water	C–S–H	calcium hydroxide

$$2C_2S + 9H \rightarrow C_3S_2H_8 + CH \quad (4.2)$$

dicalcium silicate — water — C–S–H — calcium hydroxide

The principal hydration product is a calcium silicate hydrate. The formula $C_3S_2H_8$ is only approximate because the composition of this hydrate is actually variable over quite a wide range. (In particular, the water content can vary widely, as is discussed in more detail in Section 4.2. The value here refers to the water content for the saturated condition.) $C_3S_2H_8$ is a poorly crystalline material that forms extremely small particles in the size range of colloidal matter (less than 1 μm) in any dimension. Its name, C–S–H (or C–S–H gel) reflects these properties. The properties of this hydration product will be discussed in more detail later. In contrast, calcium hydroxide is a crystalline material with a fixed composition.

Both of the previously described reactions are exothermic, which means that heat is released as the reaction progresses. This heat will raise the temperature of the concrete, unless quickly released to the surroundings. In the laboratory, hydration can be followed in a special calorimeter by monitoring the rate of heat conduction needed to keep the temperature constant. The reaction sequence is most conveniently described by reference to the calorimetric curve (see Figure 4.1) that describes the rate of heat evolution with time. The heat flow is proportional to the rate of reaction and is easily measured in the case of C_3S, which is the compound of major interest. When first mixed with water, a *period of rapid evolution of heat* (stage 1) occurs, which ceases within about 15 min. There follows a period of relative inactivity, the *induction or dormant period* (stage 2), which is the reason why portland cement concrete remains in the plastic state for several hours. Initial set occurs in 2 to 4 h, at the time C_3S has begun to

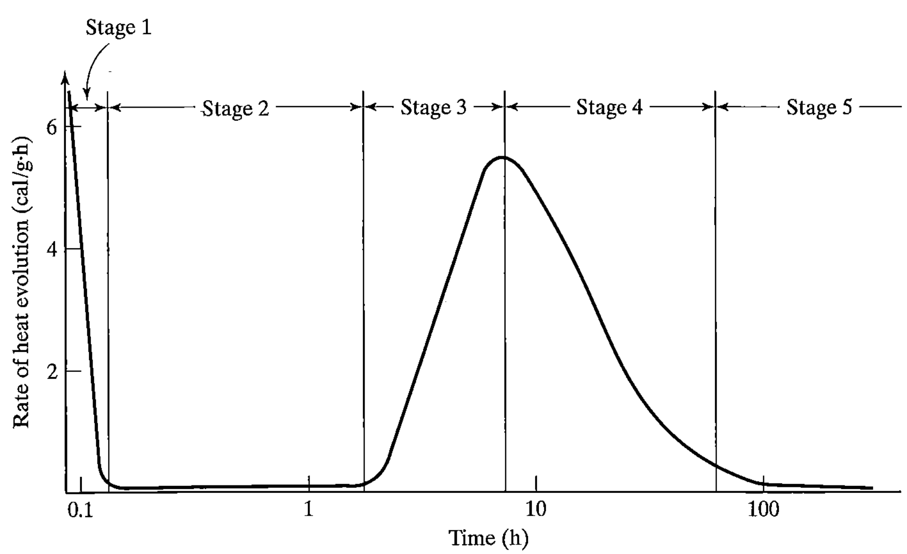


FIGURE 4.1
Rate of heat evolution during hydration of tricalcium silicate.

react again with renewed vigor at the end of the induction period. The silicate continues to hydrate rapidly, reaching a maximum rate at the end of the *acceleration period* (stage 3), which corresponds with the maximum rate of heat evolution. By this time (4 to 8 h), final set has been passed and early hardening has begun. Thereafter, the rate of reaction again slows down (stage 4) until it reaches a *steady state* (stage 5) within 18 to 36 h.

The following processes occur at each stage (Table 4.1). On first contact with water, calcium ions and hydroxide ions are rapidly released from the surface of each C_3S grain; the pH rises to over 12 within a few minutes, which indicates a very alkaline solution. This hydrolysis slows down quickly, but continues throughout the induction period. When the calcium and hydroxide concentrations reach a critical value, the hydration products, CH and C–S–H, start to crystallize from solution and the reaction of C_3S again proceeds rapidly. The induction period is apparently caused by the need to achieve a certain concentration of ions in solution before crystal nuclei form from which the hydration products grow. This is a requirement for many chemical reactions and is known as nucleation control. CH crystallizes from solution, while C–S–H apparently develops at the surface of the C_3S and forms a coating covering the grain. As hydration continues, the thickness of the hydrate layer increases and forms a barrier through which water must flow to reach the unhydrated C_3S and through which ions must diffuse to reach the growing crystals. Eventually, mass transport through the C–S–H layer determines the rate of reaction, and hydration becomes diffusion controlled. Reactions that are diffusion controlled are quite slow and become slower as the thickness of the diffusion barrier increases. Thus, hydration tends to approach 100% completion asymptotically.

C_2S hydrates in a similar manner, but is much slower because it is a less reactive compound than C_3S. The amount of heat liberated by the hydration of C_2S is also lower than it is with C_3S, and thus its calorimetric curve is not easy to measure experimentally.

TABLE 4.1 Sequence of Hydration of the Calcium Silicates

Reaction Stage	*Kinetics of Reaction*	*Chemical Processes*	*Relevance to Concrete Properties*
1 Initial hydrolysis	Chemical control; rapid	Initial hydrolysis; dissolution of ions	
2 Induction period	Nucleation control; slow	Continued dissolution of ions	Determines initial set
3 Acceleration	Chemical control; rapid	Initial formation of hydration products	Determines final set and rate of initial hardening
4 Deceleration	Chemical and diffusion control; slow	Continued formation of hydration products	Determines rate of early strength gain
5 Steady state	Diffusion control; slow	Slow formation of hydration products	Determines rate of later strength gain

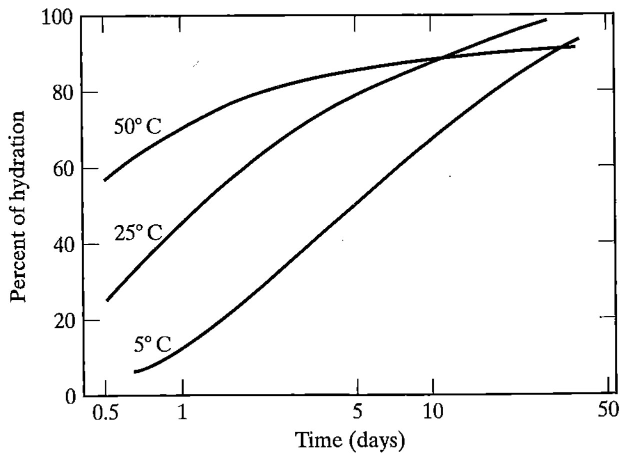


FIGURE 4.2
Effect of temperature on the hydration of tricalcium silicate.

Chemical reactions are sensitive to temperature, the rate of reaction increasing with temperature. This is also true for hydration, but the temperature dependence is related to the extent of reaction. Hydration is most sensitive to temperature through stage 3 when the reaction is chemically controlled. Once hydration is completely diffusion controlled in stage 5, it is much less temperature sensitive, although the diffusion coefficient of the hydrate barrier will vary with temperature. The overall effect of temperature is illustrated in Figure 4.2, which compares the amount of hydration of C_3S at different temperatures. The stoichiometry of hydration remains essentially the same up to about 100°C, although the composition of the C–S–H changes somewhat. At higher temperatures, which are used in autoclaving, the chemistry begins to change, as will be discussed in Chapter 12.

Tricalcium Aluminate In portland cement, the hydration of C_3A involves reactions with sulfate ions that are supplied by the dissolution of gypsum. The reactions are summarized in Table 4.2. The primary initial reaction of C_3A is

$$\underset{\text{tricalcium aluminate}}{C_3A} + \underset{\text{gypsum}}{3C\bar{S}H_2} + \underset{\text{water}}{26H} \rightarrow \underset{\text{ettringite}}{C_6A\bar{S}_3H_{32}} \qquad (4.3)$$

This calcium sulfoaluminate hydrate, whose correct name is 6-calcium aluminate trisulfate-32-hydrate, is commonly called "ettringite," which is the name given to a naturally occurring mineral of the same composition. The formula is often written $C_3A \cdot 3C\bar{S} \cdot H_{32}$. Ettringite is a stable hydration product only while there is an ample supply of sulfate available (see Table 4.2). If the sulfate is all consumed before the C_3A has completely hydrated, then ettringite transforms to another calcium sulfoaluminate hydrate containing less sulfate:

$$2C_3A + C_6A\bar{S}_3H_{32} + 4H \rightarrow 3C_4A\bar{S}H_{12} \qquad (4.4)$$

This second product is called tetracalcium aluminate monosulfate-12-hydrate, or simply monosulfoaluminate. (There is no known mineral of this composition.) It is often

TABLE 4.2 Formation of Hydration Products from C_3A

$C\bar{S}H_2/C_3A$ Molar Ratio	*Hydration Products Formed*
3.0	Ettringite
3.0–1.0	Ettringite + monosulfoaluminate
1.0	Monosulfoaluminate
<1.0	Monosulfoaluminate solid solution
0	Hydrogarnet

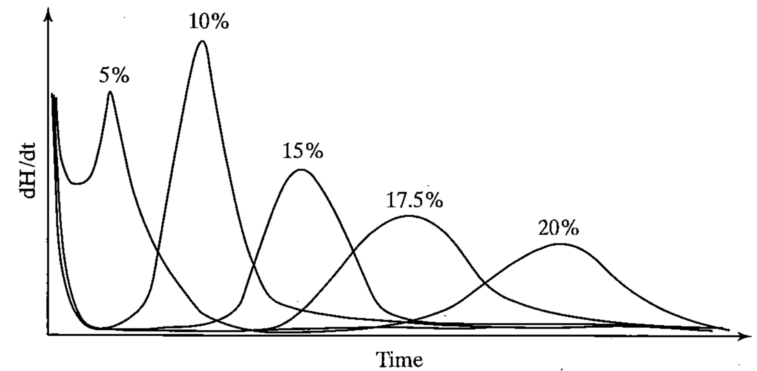


FIGURE 4.3
Rate of heat evolution during hydration of tricalcium aluminate with gypsum. (Wt. % of gypsum added is given for each curve.)

written $C_3A \cdot C\bar{S} \cdot H_{12}$. Monosulfoaluminate may sometimes form before ettringite if hydrating C_3A consumes the sulfate ions faster than they can be supplied by dissolution of the gypsum in the mix water. A certain concentration of sulfate ions is required for the formation of ettringite.

Both steps in the hydration of C_3A [Eqs. (4.3) and (4.4)] are exothermic. The formation of ettringite slows down the hydration of C_3A by creating a diffusion barrier around C_3A, analogous to the behavior of C–S–H during the hydration of the calcium silicates. This barrier is broken down during the conversion to monosulfoaluminate and allows C_3A to react rapidly again. Thus, the calorimeter curve for hydrating C_3A (Figure 4.3) looks qualitatively much like the curve for C_3S, although the underlying reactions are quite different and the amount of heat evolved is much greater. The first heat peak is completed in 10 to 15 min, but the time at which the second occurs depends on the amount of sulfate available. The more gypsum there is in the system, the longer the ettringite will remain stable. Conversion to monosulfoaluminate will occur in most cements within 12 to 36 h, after all the gypsum has been used to form ettringite.

The formation of monosulfoaluminate occurs because in most cements there is not sufficient gypsum necessary to form ettringite from all the available aluminate ions. When monosulfoaluminate is brought into contact with a new source of sulfate ions, then ettringite can be formed once again:

$$C_4A\bar{S}H_{12} + 2C\bar{S}H_2 + 16H \rightarrow C_6A\bar{S}_3H_{32} \tag{4.5}$$

This potential for reforming ettringite is the basis for sulfate attack of portland cements when exposed to an external supply of sulfate ions (see Chapter 18).

Gypsum is added to curb the vigorous initial reaction of C_3A with water, which can lead to flash set (Chapter 9), due to the rapid formation of calcium aluminate hydrates:

$$C_3A + 21H \rightarrow C_4AH_{13} + C_2AH_8 \tag{4.6}$$

These hydrates are not stable and later convert to C_3AH_6 (hydrogarnet):

$$C_4AH_{13} + C_2AH_8 \rightarrow 2C_3AH_6 + 9H \tag{4.7}$$

This conversion is exactly the same as that found in high-alumina cement, but occurs so rapidly (because the high rate of heat liberation causes a large temperature rise) that pure C_3A pastes do not develop substantial strength. Even with gypsum present, the formation of sulfate-free hydrates may not be entirely suppressed if the C_3A is very reactive (although not enough will form to cause flash set), and small amounts of hydrogarnet may be found in a hydrated cement.

When quite small amounts of gypsum are present, there may still be unreacted C_3A present when all of the ettringite has been converted to monosulfoaluminate. In such cases, a solid solution between $C_4A\bar{S}H_{12}$ and C_4AH_{13} is formed, the two hydrates having the same crystal structure. This solid solution is written $C_3A(C\bar{S},CH)H_{12}$, and the reaction is

$$C_4A\bar{S}H_{12} + C_3A + CH + 12H \rightarrow 2C_3A(C\bar{S},CH)H_{12} \tag{4.8}$$

Ferrite Phase C_4AF forms similar hydration products to C_3A, with or without gypsum. The reactions are slower and involve less heat; C_4AF seldom hydrates rapidly enough to cause flash set, and gypsum retards C_4AF hydration even more drastically than it does C_3A. Changes in the composition of the ferrite phase affect only the rate of hydration; as the iron content is raised, hydration becomes slower. Iron oxide apparently plays the same role as alumina during hydration (i.e., F can substitute for A in the hydration products). As can be seen in Eqs. (4.9a) and (4.9b), there is insufficient lime to form the calcium sulfoaluminates unless amorphous hydrous oxides of iron or aluminum form also:

$$C_4AF + 3C\bar{S}H_2 + 21H \rightarrow C_6(A,F)\bar{S}_3H_{32} + (F,A)H_3 \tag{4.9a}$$

$$C_4AF + C_6(A,F)\bar{S}_3H_{32} + 7H \rightarrow 3C_4(A,F)\bar{S}H_{12} + (F,A)H_3 \tag{4.9b}$$

In these equations, the use of a formula such as $C_6(A,F)\bar{S}_3H_{32}$ indicates that iron oxide and alumina occur interchangeably in the compound, although the A/F ratio need not be the same as that of the parent compound, and the formula (F,A) indicates that the product is richer in iron.

Practical experience has shown that cements low in C_3A, but high in C_4AF are much more resistant to sulfate attack. This means that the formation of ettringite from monosulfoaluminate, as in Eq. (4.5), does not occur. It has not been established why this is so; it may be that an iron-substituted monosulfoaluminate cannot react to form ettringite. Alternatively, the presence of the amorphous $(F,A)H_3$ may in some way prevent the reaction described in Eq. (4.5) from occurring.

Special Aluminate Compounds The formation of ettringite from calcium fluoroaluminate or sulfoaluminate also proceeds according to specific chemical reactions. As in the hydration of C_3A, a deficiency of sulfate ions ensures the eventual formation of monosulfoaluminate in most cases. With regulated-set cement, this occurs after several hours when the hemihydrate has largely reacted and sulfate is supplied solely by less-soluble anhydrite. In the case of expansive cements, conversion does not take place until after expansion is completed, which may be several weeks, although in some Type E-1(K) cements ettringite remains the stable phase. Whenever substantial amounts of monosulfoaluminate are present, the possibility exists for a reversion to ettringite according to Eq. (4.5). Such cements are therefore prone to sulfate attack and should not be exposed to sulfate-bearing environments.

Hydration of Portland Cement

Kinetics The rate of hydration during the first few days is in the approximate order $C_3A > C_3S > C_4AF > C_2S$. It must be remembered, however, that none of these compounds will hydrate at exactly the same rate in different cements because their reactivities will be affected by differences in fineness and the rate of cooling of the clinker. There will be additional factors, such as the presence of impurities and the presence of the other cement compounds. For example, alite and belite hydrate faster than do pure C_3S and C_2S because of the impurity atoms contained in the structure. The hydration of C_3A and the ferrite phase are affected by impurity oxides.

Compound Interactions The assumption made earlier that the cement compounds hydrate independently is a reasonable one for most purposes, but is not entirely true. For example, C_3A and C_4AF both compete for sulfate ions, but the more reactive C_3A will consume more sulfate than does C_4AF. The effect is to increase the reactivity of C_4AF, since it forms less ettringite than would be expected. Gypsum increases the rate of hydration of the calcium silicates, which also compete for sulfate during hydration. C–S–H incorporates significant amounts of sulfate, and also alumina and iron, into its structure. It has been estimated that the quantity of sulfoaluminates that forms in a paste may be less than half the theoretical quantity calculated from the compound composition of the cement.

Another example is the existence of an optimum gypsum content for the development of maximum strength in a cement paste. One explanation is that too high a gypsum content results in the formation of excessive amounts of ettringite after the paste has hardened, causing unrestrained expansion and disruption of the paste microstructure, whereas too low a gypsum content allows the monosulfoaluminate-solid solution [Eq. (4.8)] to form before the end of stage 2 of C_3S hydration, so that the resulting consumption of lime prevents the nucleation of the hydration products of C_3S, delaying stage 3. Another explanation suggests that gypsum accelerates C_3S hydration, but at the same time lowers the intrinsic strength of C–S–H due to the presence of sulfate ions in its structures. Since there is a different optimum gypsum content for maximum

strengths at different ages, both explanations may contribute to the phenomenon. The interaction between C–S–H and sulfate ions may help explain the observation that there is also an optimum gypsum content for minimum drying shrinkage, which is not the same as for strength.

Heat of Hydration All hydration reactions are accompanied by release of heat. The heat of hydration (ΔH)[1] is a measure of the amount of heat evolved for each unit mass of anhydrous compound that has reacted. It represents the residual energy in the system after the redistribution of energy has occurred with the breaking and making of chemical bonds during hydration. ΔH is enthalpy, a thermodynamic quantity that can either be determined by calculation or measured experimentally, as described in Chapter 3 (see Table 4.3).

We have already discussed the practical importance of the evolution of heat that accompanies hydration. It is possible in theory to calculate the heat of hydration of portland cement at any given time if the compound composition, the amount of hydration that has occurred for each compound, and ΔH for the reaction are known. The contributions to heat evolution are dominated by C_3S and C_3A, particularly at early hydration times. A typical calorimetric curve for cement is shown in Figure 4.4; generally, the contributions of both C_3S and C_3A can be distinguished.

TABLE 4.3 Heats of Hydration of the Cement Compounds

Reaction	ΔH (J/g) for Complete Hydration[a]			
	Pure Compounds		Clinker[b,d]	Cement[c,d]
	Calculated	Measured	Measured	Measured
$C_3S \rightarrow$ C–S–H + CH	~380	520	570	490
$C_2S \rightarrow$ C–S–H + CH	~170	260	260	225
$C_3A \rightarrow C_4AH_{13} + C_3AH_8$	~1160	–	–	–
$\rightarrow C_3AH_6$	900	880	840	–
$\rightarrow$ ettringite	1670	1670	–	–
$\rightarrow$ monosulfoaluminate	1150	1140	–	1170
$C_4AF \rightarrow C_3(A,F)H_6$	420	420	335	–
$\rightarrow$ monosulfoaluminate	–	–	–	380
$\rightarrow$ ettringite	730	–	–	–

[a]These values should be negative since they refer to exothermic reactions, but they are customarily written without the negative sign.
[b]One-year-old pastes of ground clinker (no added gypsum).
[c]One-year-old pastes assumed to be completely hydrated.
[d]Individual contributions determined by multiple linear regression analysis.

[1]Heat of hydration is the enthalpy of the reaction. It is normally expressed as kJ/mole, but since cement is a multicomponent material the mole concept cannot be used.

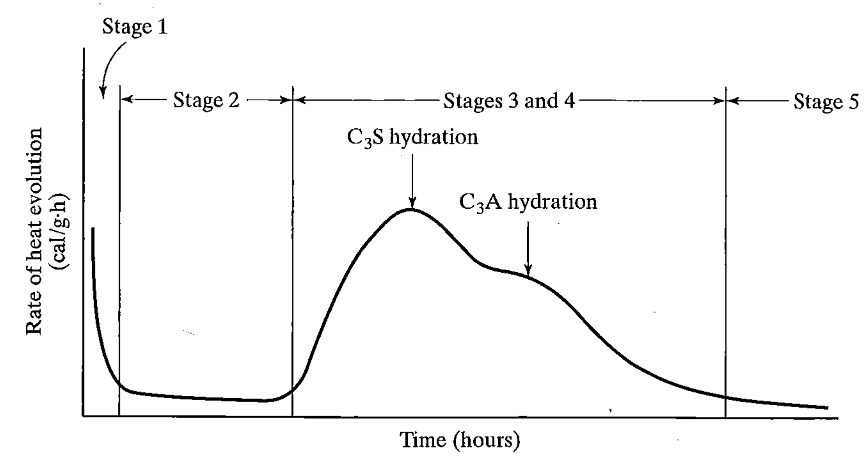


FIGURE 4.4
Rate of heat evolution during hydration of portland cement.

4.2 PROPERTIES OF THE HYDRATION PRODUCTS

The various hydration products that are formed in hydrated cement pastes have quite diverse properties, and the behavior of each compound will contribute to the overall behavior of the paste. In this section, we will briefly review the properties of the individual hydration products; a review of their physical interrelationships can be found in the section on microstructure. The important properties are summarized in Table 4.4.

C–S–H

Compositional Variations This calcium silicate hydrate is not a well-defined compound. The formula $C_3S_2H_8$ is only an approximate description, as the stoichiometry is quite variable. The C/S ratio is generally higher than 1.5, varying between 1.5 and 2.0. It depends on many factors: the age of the paste, the temperature of hydration, the *w/c* ratio, and the amount and kind of impurity oxides that can be incorporated into the product.

The water content varies even more drastically and depends on the extent of drying as well as the factors just mentioned. Unlike most hydrated compounds, C–S–H has no definite hydration states, and when C–S–H is dried, water is not lost at discrete partial pressures of water vapor (relative humidities) to give specific hydrates. Rather, there is a continuous loss of water as the relative humidity is lowered from 100% RH to strong drying in vacuum or on heating. This is because the water associated with C–S–H exists in several different states. Because of the continuous loss of moisture with decreasing water vapor pressures, it is necessary to define standard drying conditions when determining water contents. The one most widely adopted is *D-drying* (see Table 4.5), which is the most rigorous vacuum drying condition. Under these conditions, loss of residual moisture is very slow, and several days drying is required, even when the paste is ground to a fine powder. Engineers find oven drying at 105°C to

TABLE 4.4 Summary of Properties of the Hydration Products of Portland Cement Compounds

Compound	*Specific Gravity*	*Crystallinity*	*Morphology in Pastes*	*Typical Crystal Dimensions in Pastes*	*Resolved by*[a]
C–S–H	2.3–2.6[b]	Very poor	Spines; Unresolved morphology	1 × 0.1 μm (Less than 0.01 μm thick)	SEM, TEM
CH	2.24	Very good	Nonporous striated material	0.01–0.1 mm	OM, SEM
Ettringite	~1.75	Good	Long slender prismatic needles	10 × 0.5 μm	OM, SEM
Monosulfo-aluminate	1.95	Fair–good	Thin hexagonal plates; irregular "rosettes"	1 × 1 × 0.1 μm	SEM

[a]OM, optical microscopy; SEM, scanning electron microscopy; TEM, transmission electron microscopy.
[b]Depends on water content.

be more convenient and to be satisfactory for most purposes. It gives about the same water content and is much quicker than vacuum drying, water loss being complete after 24 h of heating. The water content of C–S–H determined for D-dried material corresponds to $C_3S_2H_3$. However, since C–S–H is in (or close to) the saturated state in hardened cement paste, the formula in Eqs. (4.1) and (4.2) should be used when calculating volume relationships.

Physical Behavior Because of its variable composition, C–S–H is not a well-crystallized material. In fact, it is essentially amorphous and develops as a mass of extremely small irregular particles of indefinite morphology. The particles are so small that they can be studied only by electron-optical techniques, and even then cannot be completely resolved. Observed morphologies, such as irregular spines (aciculae) or "lace-like" structures are artifacts of drying. As a consequence of this very finely divided state of C–S–H hydrated cement pastes have very high surface areas. Measurements indicate that most C–S–H preparations formed at ambient temperatures have surface areas close to 400 m^2/g. Such high surface areas are characteristic of colloids.

TABLE 4.5 Standard Drying Conditions Used to Determine Water Contents of C–S–H and Hydrated Cement

Type of Drying	*Temperature*	*Conditions*
D-drying	Ambient	Vacuum over dry ice at –78°C
Oven drying	105°C	Heating in oven at ambient pressure
Solvent replacement	Ambient	Soaking in methanol or isopropanol for several days

Model of C–S–H Structure Because of its amorphous character, compositional variability, and poorly resolved morphology, C–S–H is a difficult material to study. Interpretation of experimental data is still a subject of active debate. Various conceptual models have been proposed that emphasize different aspects of the structure to explain observed experimental results. No one model can be considered to be a "correct" description in any absolute sense, but a good model will provide additional insights into the behavior of a material and predict hitherto unrecognized properties. Models must be modified as new data are obtained. The following simplified description is based on several models.

C–S–H can be considered to have a degenerate clay structure, by which it is meant that it is based on a layered structure (see Figure 4.5). A well-crystallized clay mineral has the structure shown in Figure 4.5a. It can be thought of as being composed of layers of bread and filling to make a generous club sandwich. The "bread" is composed of silicoaluminate sheets that are stacked in a specific orientation; the "filling" is made up of metal ions that hold the sheets together with comparatively weak electrostatic attractions between positive charges on the metal ions and residual negative charges on the sheets. Water is also present between the layers. In some clays, the layers can be expanded to accommodate additional water, thereby expanding the crystal. Loss of *interlayer water* on drying allows the layers to collapse again and the crystal to

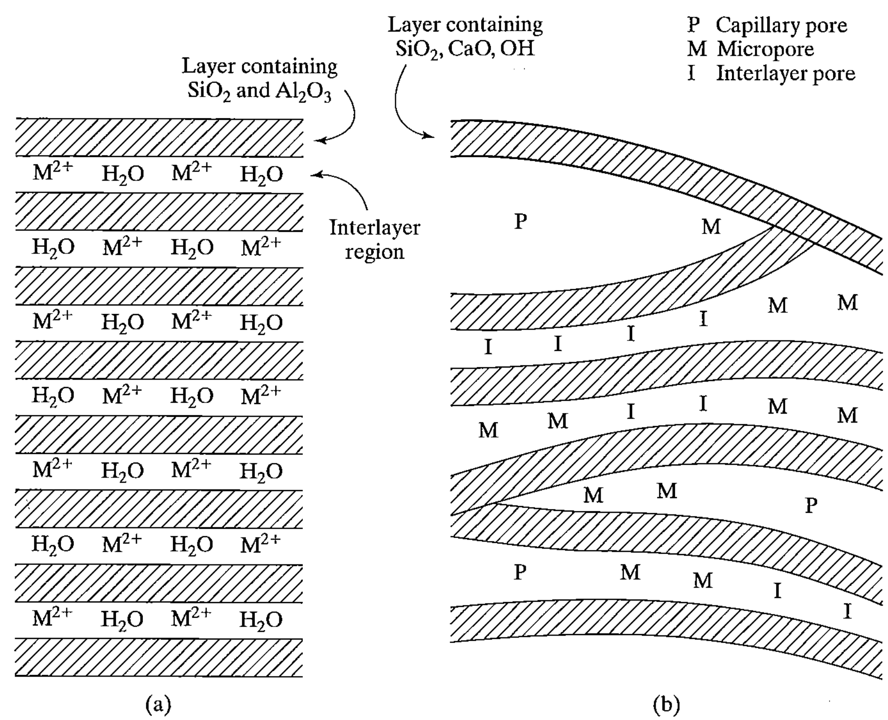


FIGURE 4.5

Schematic model of C–S–H in cement paste: (a) well-crystallized clay mineral; (b) poorly crystallized C–S–H.

contract. This results in the clays having large volume changes on wetting and drying, and C–S–H behaves similarly.

In C–S–H the "bread" is calcium silicate sheets, and the "filling" is additional calcium ions and water molecules. Unlike a well-crystallized clay mineral, however, the sheets are distorted and randomly arranged (owing to random variations in their composition) so that they do not fit together neatly. This is shown schematically in Figure 4.5b. As a result, the spaces between the sheets are irregular and vary considerably in their dimensions. (Another analogy is to visualize a clay mineral as a stack of sheets of copy paper. If these sheets are crumpled up one by one, smoothed out, and restacked, they will not lie perfectly flat and will be more randomly arranged with respect to each other. This is the C–S–H structure.)

The space between the calcium silicate sheets is the intrinsic porosity of C–S–H. We can distinguish three kinds of pores: interlayer pores (I), micropores (M), and isolated capillary pores (P). Capillary pores are spaces in which water can behave as bulk water and menisci are created as the pores are filled or emptied. In micropores, the adjoining surfaces are so close together that water cannot form menisci and consequently has a different behavior from bulk water. Water in micropores acts to keep the layers apart by exerting a *disjoining* pressure. The disjoining pressure depends on the relative humidity and disappears below 50% RH. When the sheets forming the micropores approach closely in a specific orientation, they may form clay-like interlayer spaces (I) that bond the sheets together at this point. Interlayer bonding can be regarded as a special case of van der Waals' bonding. In addition, sheets will from time to time be bonded directly by strong ionic-covalent bonds, which do not involve the weaker interlayer bonding.

Interaction with Water Water plays a very important role in the structure and behavior of C–S–H because of its strong interaction with surfaces (through hydrogen bonding) and its small molecular size. Thus, it can access all surfaces in the material including the interlayer region, which is inaccessible to most other molecules. Like other colloids, C–S–H is a material with a high potential energy because of the additional energy associated with its high surface area and poor crystallinity. The material will respond to external conditions in a way to lower its potential energy and increase the ionic-covalent bonding between particles—that is, by lowering its surface area and enhancing atomic ordering. For example, during prolonged moist curing, or curing at higher temperatures, the C–S–H structure becomes more stable and more resistant to subsequent environmental changes, like drying.

Water is held within C–S–H in a variety of ways, ranging from bulk water in isolated capillary pores, through water physically adsorbed on surfaces or between surfaces (micropores) and water structurally associated with the solid (interlayer water), to hydroxyl water in the solid lattice. There appears to be no sharp distinction between the various forms, but a gradual transition between them. Thus, there are no universally accepted experimental methods for distinguishing between the different kinds of water. As water is removed from C–S–H, rearrangement of the particles or layers is possible. For example, loss of water from capillary pores is believed to induce compressive stress on the system due to surface-tension effects, while loss of water between the layers or from surfaces changes the potential for bonding: either ionic-covalent,

van der Waals', or interlayer. There is evidence to suggest that, when it is first formed, fully saturated C–S–H may have a surface area exceeding 750 m^2/g, but that this is greatly reduced by the extent and method of drying. Changes in bonding may cause permanent changes in the structure; exposure of moist C–S–H to higher temperatures may give similar changes. Different methods of drying cement paste that are commonly used are summarized in Table 4.5. Drying by solvent replacement gives rise to the least microstructural change.

Calcium Hydroxide

In contrast to C–S–H, calcium hydroxide is a well-crystallized material with a definite stoichiometry. Crystals large enough to be seen with the naked eye can sometimes grow inside voids formed in concrete, and these have a distinctive hexagonal tabular morphology. Within the body of the paste, the crystals do not grow so large and have a less distinctive form, but often can still be seen under a light microscope. For reasons discussed in Section 4.3, calcium hydroxide does not form homogeneous crystals in a cement paste.

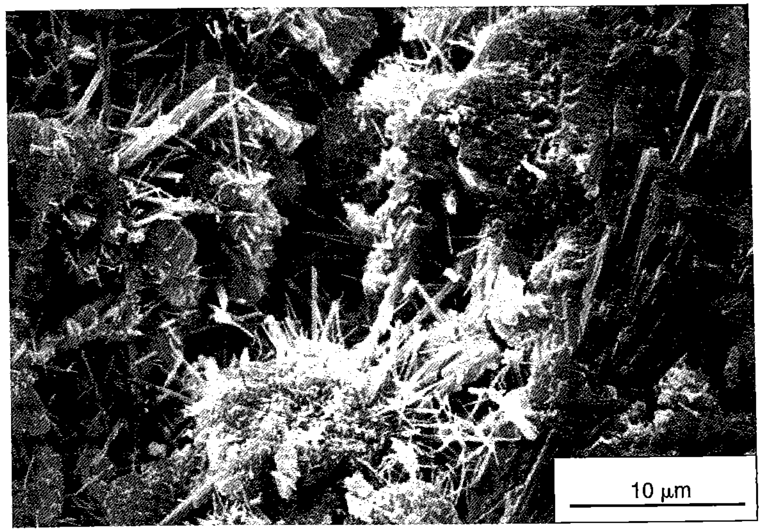
Calcium Sulfoaluminates

Ettringite also crystallizes as hexagonal crystals, but in the form of needles with a much greater aspect ratio (ratio of length to diameter). The exact morphology depends on the available space and supply of ions for crystal growth. In the common portland cements, ettringite crystals are generally seen as long slender needles (Figure 4.6a), typically $10 \times 0.5\ \mu m$. In expansive cements, where large quantities form rapidly, the needles are shorter and thicker and well intergrown. Generally, the individual crystals are not large enough to be seen under the optical microscope. However, in some concretes that have deteriorated from sulfate attack, large clusters of ettringite needles radiating from a center can often be seen on petrographic examination. Crystal masses of ettringite can frequently be observed filling in cracks or voids in mature concretes. Ettringite loses considerable amounts of its crystal water on drying.

When first formed, monosulfoaluminate or its solid solution tends to form clusters or "rosettes" of irregular plates. Later, these tend to grow into well-developed, but very thin, hexagonal plates (Figure 4.6b) that are too small to be seen by the optical microscope. In pure systems, these plates are well crystallized, but in portland cement pastes the incorporation of impurities decreases the degree of crystallinity to some extent. $C_4A\bar{S}H_{12}$, C_4AH_{13}, and C_2AH_8 are all structurally related and form the same kinds of crystals, and all lose water on drying. Their crystal structure is also based on a layer arrangement analogous to the clay structure. Layers of positively charged calcium hydroxyaluminate are balanced by anions (OH^-, SO_4^{2-}, $Al(OH)_4^-$, etc.) between the layers together with water molecules.

4.3 MICROSTRUCTURE OF HYDRATED CEMENT PASTES

Although the properties of the hydration products will clearly influence the properties of the hydrated cement paste, the behavior of hardened cement paste cannot be properly understood without an appreciation of how the hydration products fit together to



(a)



(b)

FIGURE 4.6

SEM micrographs of calcium sulfoaluminate hydrates in hydrated cement paste: (a) ettringite as first formed in portland cement paste (note striated CH crystal on right hand side); and (b) monosulfoaluminate formed from C_3A.

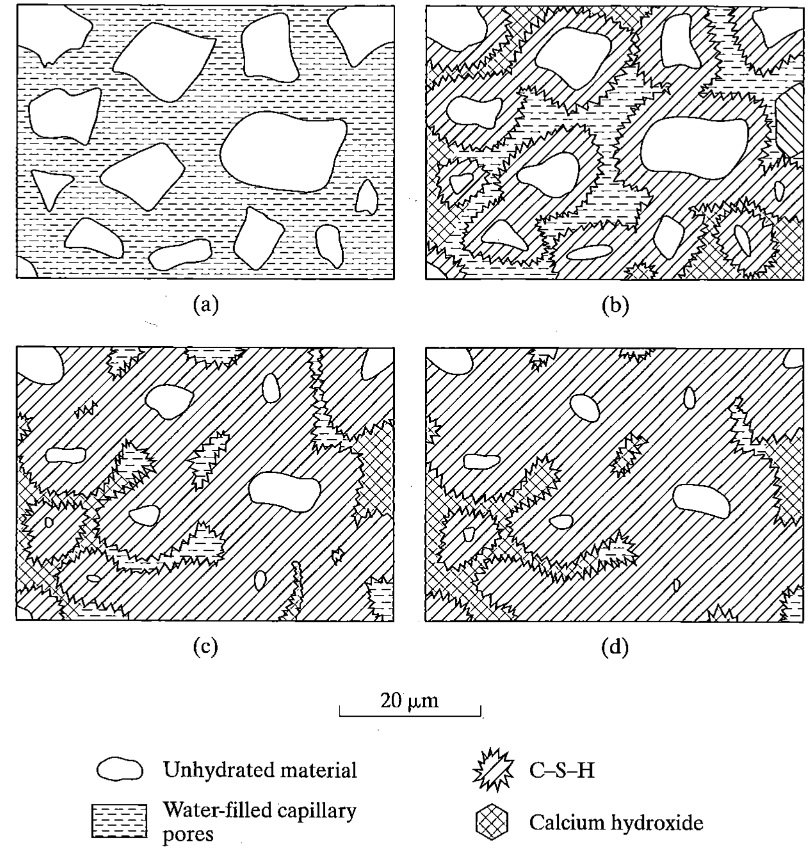
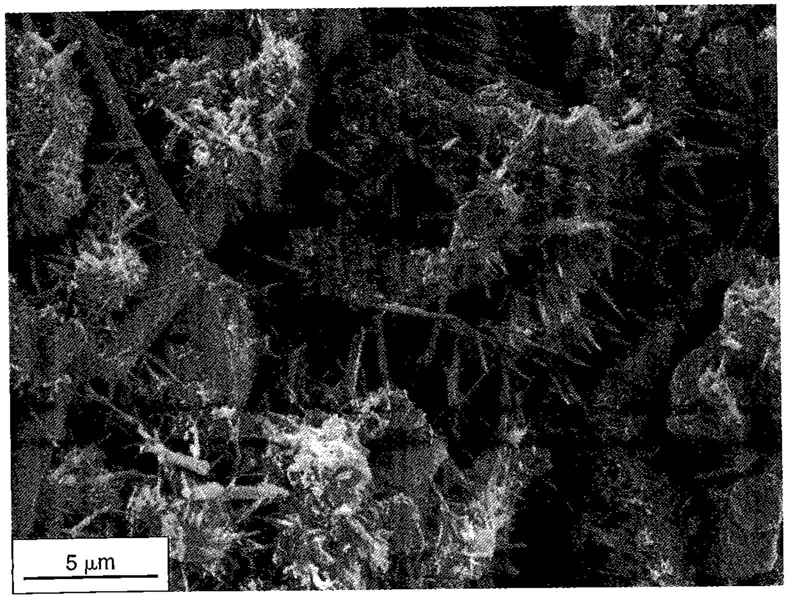


FIGURE 4.7

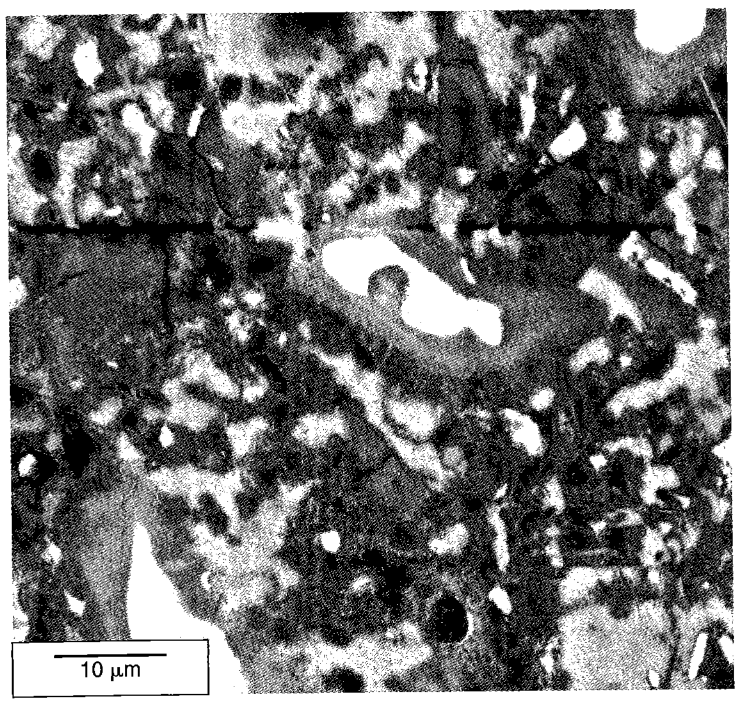
Schematic outline of microstructural development in portland cement pastes: (a) initial mix; (b) 7 days; (c) 28 days; and (d) 90 days. (Calcium sulfoaluminates are included as part of C–S–H for simplification, although they crystallize as separate phases.)

form the cementing matrix. The description of microstructural development that follows is based on SEM observations. Figure 4.7 shows schematically the sequence of structure formation as hydration proceeds. This involves the replacement of water that separates individual cement grains in the fluid paste (Figure 4.7a) with solid hydration products that form a continuous matrix and bind the residual cement grains together over a period of time, as illustrated in Figure 4.7(b–d). This happens because the hydration products occupy a greater volume than the original cement compounds, owing to their lower specific gravity (~2.0 compared to 3.2).

The microstructures of young and mature hydrated cement paste are shown in Figure 4.8. Figure 4.8a is a fracture surface that reveals the morphology and arrangement of all the components. Figure 4.8b is a randomly cut and polished section through the paste and is a more representative view of the overall microstructure, which shows a high degree of complexity.



(a)



(b)

FIGURE 4.8

SEM micrographs of hydrated cement paste: (a) Fracture surface of paste with less than three days' hydration, showing ettringite needles growing into water-filled pore space, platey crystals of CH, and C–S–H with indeterminate morphology. (Courtesy of Paul Stutzman, National Institute of Standards and Technology.) (b) Cut and polished cross section of a 28-day-old paste, showing large bright unhydrated cores of large grains surrounded by darker C–S–H (phenograins). The "groundmass" is a mixture of C–S–H, light-gray CH, and black capillary pores. [Photograph from D. Darwin, M. N. Abou-Zeid, and K. W. Ketchum.

C–S–H

Since C–S–H makes up over one half of the volume of a hydrated paste, it is the most important component. Two forms of C–S–H can be identified in the microstructure.

Early Product C–S–H During early hydration, C–S–H grows out from the particle surface into the surrounding water-filled space in the form of a low-density arrangement of thin sheets. This form of outer product C–S–H (early product) has a higher microporosity and on drying rearranges to a variety of morphological forms (such as seen in Figure 4.8a) and coarser porosity. This C–S–H also contains a high level of impurities (aluminum, sulfate, alkalis) and is probably admixed with $C_4A\bar{S}H_{12}$ at the nanometer level. This rather variable component of C–S–H also has been called both the "groundmass" and "undesignated product" (UDP).

Late Product C–S–H Once hydration has become diffusion controlled, C–S–H forms primarily as a denser coating around the hydrating cement grains, referred to as either late or "inner" product. These coatings form the diffusion barrier during later hydration and thicken with time, growing inwards as well as outwards. The coatings maintain the shape of the original grains and are clearly seen in Figure 4.8b surrounding unhydrated residues. These prominent features have been called "phenograins," although the term has been used to describe any significant feature that is distinct from the groundmass regardless of their composition. This late product C–S–H is denser, has less impurities, and is more resistant to physical change on drying. This C–S–H has a more stable arrangement, and nanometer-sized crystallites have been observed, some of which consist of CH. The proportion of late product C–S–H increases as hydration increases or the *w/c* decreases.

Calcium Hydroxide

Calcium hydroxide crystals occupy about 20 to 25% of the paste's solid volume. During Stage 3 hydration of C_3S, many calcium hydroxide crystals nucleate and grow within the capillary pore space. Their size is such that they are readily studied by optical microscopy, which shows clearly how the isolated crystals gradually spread throughout the paste (Figures 4. 9a and b). The characteristic striated appearance of the crystal is a consequence of the way the crystal fractures within the paste. Calcium hydroxide will only grow where free space is available. If it is impeded by another calcium hydroxide crystal, it may stop growing or grow in another direction. When it encounters a hydrating cement particle, it will grow right around it, as can be seen in Figure 4.9c.

Often a calcium hydroxide crystal may completely engulf a cement particle, which increases the effective volume that the crystal occupies in the paste. More importantly, calcium hydroxide grown *in situ* may behave quite differently from pure crystals. Calcium hydroxide may vary in morphology, being found as small equidimensional crystals; large flat, platy crystals; large, thin, elongated crystals; and all variations in between. Morphology is particularly affected by admixtures and by the temperature of hydration.

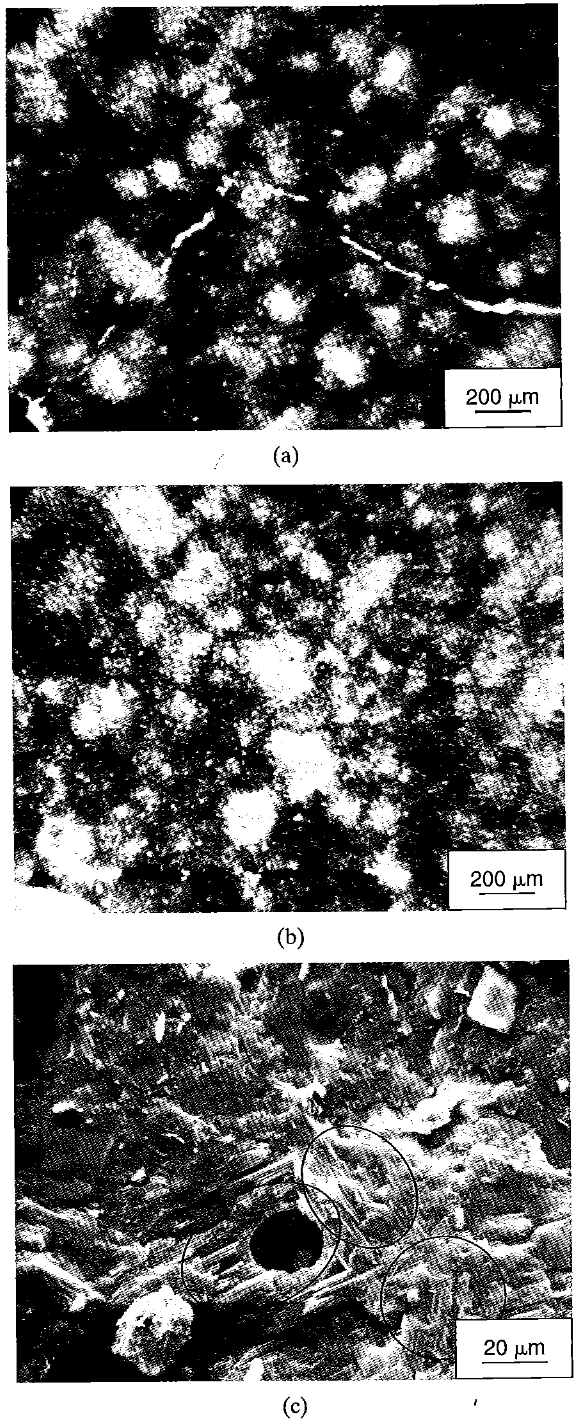


FIGURE 4.9

Calcium hydroxide in hydrated cement pastes: (a) optical micrograph, 7 days hydration; (b) optical micrograph, 15 days hydration; (c) SEM micrograph, 64 days; circles enclose large CH crystals.

Calcium Sulfoaluminates

The calcium sulfoaluminates are a relatively minor constituent of a mature paste, making up only 10–15% by solid volume. Thus, they play a correspondingly minor role in the microstructure of the hydrated cement paste (although not necessarily in its properties) and are therefore omitted in Figure 4.7. Ettringite is seen in SEM micrographs of very young pastes as needles growing into the capillary pores between cement grains (Figure 4.8a). These later convert to the platey morphology of the monosulfoaluminate (Figure 4.6b). Both phases are well dispersed throughout the paste, and some exist as a mixture with C–S–H on the nanometer level.

Unhydrated Residues

Unhydrated residues of large cement grains may persist even in well-hydrated cements. Occlusion by calcium hydroxide may preclude complete hydration, or there may be insufficient free space within the paste (Section 4.3).

Porosity

Porosity is another major component of the microstructure. We have been emphasizing the decrease in capillary porosity during hydration, as seen by SEM observations, but pore size distribution is also of importance. However, measurement and classification of pore sizes is complicated by experimental difficulties and interpretation. There are two main problems. First, the paste must be dried in order to make measurements, and drying almost certainly changes the pore structure. Second, a definite geometrical shape of pore must be assumed while SEM micrographs show that the pores form a network of very irregularly shaped spaces. Nevertheless, a useful assessment of pore structure can be obtained.

Classification A classification of porosity in cement paste is given in Table 4.6; it can be seen that there is an enormous range of pore sizes, from 10 μm to less than 0.0005 μm (0.5 nm) in diameter. Thus, the water that occupies the pores plays many different roles. It is useful to make a distinction between *capillary pores* and *gel pores* in a hydrated paste. The capillary pores are the remnants of water-filled space that exists between the partially hydrated cement grains; the gel porosity is regarded as an intrinsic part of the C–S–H. The porosity seen in SEM micrographs is capillary

TABLE 4.6 Classification of Pore Sizes in Hydrated Cement Pastes

Designation	*Diameter*	*Description*
Capillary Pores	10,000–50 nm (10–0.05 μm)	large capillaries (macropores)
	50–10 nm	medium capillaries (large mesopores)
Gel Pores	10–2.5 nm	small isolated capillaries (small mesopores)
	2.5–0.5 nm	micropores
	$\leq$ 0.5 nm	interlayer spaces

TABLE 4.7 Influence of Pore Sizes on Properties of the Hydrated Paste

Category	*Role of Water*	*Properties Affected*
Macropores	Behaves as bulk water	Permeability; diffusivity
Large mesopores	Small surface tension forces generated	Permeability in the absence of macropores; shrinkage above 80% RH
Small mesopores	Large surface tension forces generated	Shrinkage between 80% RH and 50% RH
Micropores	Strongly adsorbed water; no menisci form	Shrinkage at all RH; creep

porosity; the gel porosity cannot be resolved by the SEM and is included in the volume occupied by C–S–H. In mature pastes, the bulk of the porosity resides within the C–S–H. The size division between capillary and gel porosity is to a large extent arbitrary, as the spectrum of pore sizes is continuous. It should also be noted that gel pores include small capillary pores if we use the more accurate definition that a capillary pore is one in which capillarity effects can occur (i.e., a meniscus can form). However, the capillary pore system is the interconnected network of pores through which bulk water flow and ionic diffusion occur relatively easily.

As can be seen in Table 4.7, porosity over the whole size range of pores has an influence on paste properties. Yet it is difficult to get an exact assessment of pore-size distributions because no one measurement encompasses the whole size range and because it is difficult to interpret experimental data. Thus, comparisons of porosity should be made with care.

Measurement There are two main methods that are used to measure the pore size distribution of hardened cement paste: mercury intrusion porosimetry and physical adsorption of gases. Mercury porosimetry involves forcing mercury into the pore system of the paste by the application of external pressure (intrusion). Mercury does not wet the paste, and force is needed to overcome surface effects. The pressure required is inversely proportional to the pore radius. In the adsorption method, pores are filled by a condensing vapor (gas) through capillary condensation, which is a spontaneous process controlled by surface forces.

Mercury porosimetry gives a better appreciation of the capillary pore system, whereas adsorption studies give a better measure of the gel-pore system: small mesopores and micropores. Although the two methods have overlapping size ranges, they do not agree very well. There are several reasons for this. Firstly, the pastes have to be strongly dried before measurements are made. Physical changes in C–S–H affect the pore-size distributions, particularly in the micro and mesopore range. Secondly, the two physical processes are quite different. Mercury must fill pores from the outside by flowing through the pore system so that the pore-size distribution is really a profile of the size of entries controlling access to pore volumes. This is a percolation process and requires an interconnected network. Capillary condensation takes place simultaneously at all points in the paste where the required conditions are satisfied. Thirdly, in both

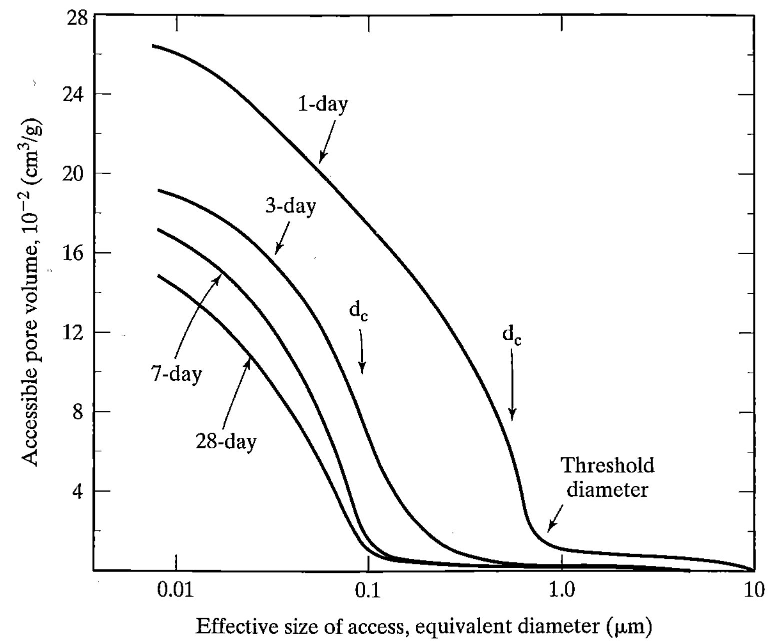


FIGURE 4.10
Mercury intrusion porosimetry curves for portland cement pastes.

cases a simple pore geometry must be assumed, although SEM examinations reveal highly irregular shapes.

Figure 4.10 shows a series of mercury porosimetry curves for pastes hydrated for different times. It can be seen that not only is the total volume of accessed pores reduced, but also the fineness of the pore system increases. The volume of macropores is rapidly reduced, but the distribution of smaller pores is less affected. A similar set of curves is found when the *w/c* ratio is gradually reduced and measurements made at the same degree of hydration. The first inflection point, the threshold diameter, indicates the onset of percolation. The point of steepest slope [$dV/d(\ln d)$] represents the mean size of pore entryways that allows maximum percolation throughout the pore system, and is called the "continuous (or critical) pore diameter" (d_c).

Pore Solutions The fluid contained in capillary pores is not pure water, but an ionic solution that is in equilibrium with the hydrated pastes. Its composition can be determined by squeezing fluid out of the paste under a high confined pressure. The solution in paste made with pure calcium silicates is essentially saturated calcium hydroxide [approximately 1.2 g/l, or 0.02 M (pH $>$ 12)]. In cement pastes, the solution chemistry is altered by the presence of gypsum and soluble alkalis, particularly when potassium sulfate (arcanite) is present (see Figure 4.11). At the end of the induction period, calculations have indicated that maximum saturation with respect to calcium hydroxide is still achieved. Once the sulfate is fully consumed by the formation of ettringite, the pore fluid is a solution of alkali hydroxides, whose concentration is controlled by the

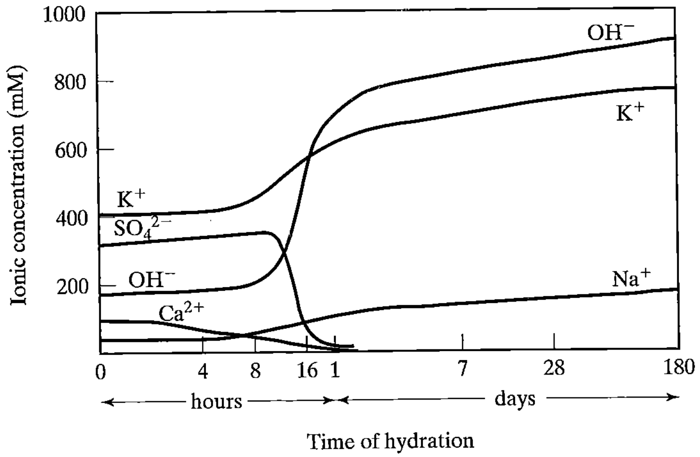


FIGURE 4.11

Typical changes in pore solution composition in hydrating cement paste with a moderately high alkali content. (Courtesy of Sidney Diamond, Purdue University.)

alkali content of the cement. With low alkali cements, it is in the vicinity of 0.2M alkali hydroxide (pH ~ 13); while for high alkali cements, it exceeds 0.5M (pH > 13.5).

Interfacial Transition Zone

The microstructure of the hydrated cement paste is highly modified in the vicinity of embedded materials: aggregates, fibers, and reinforcing steel. This modified volume is called the *interfacial transition zone* (ITZ), and in some cases, it can be easily identified by the concentration of oriented calcium hydroxide crystals in the vicinity of the foreign surface. The common features of all ITZs are the enhanced porosity and the reduction of unreacted cement near the surface of the embedment. Both are due to the inability of the cement particles to pack efficiently around the embedment, a well-known phenomenon called the "wall effect." This raises the local *w/c* ratio at the interface, which can be further increased by localized bleeding. Profiles of the ITZ composition, shown in Figure 4.12, are used to define the extent of the ITZ, which is usually 20–40 μm in width.

The detailed structure of the ITZ can be quite complex and variable; it does not necessarily have a uniform thickness and may be entirely absent at some parts of the surface. The ITZ thickness is therefore a statistical quantity. Its extent depends on a variety of factors, such as the size and shape of the aggregate (or fiber) and its volume fraction, the *w/c* ratio, the mixing and placing procedures, etc. Within the free space close to the surface, crystals of calcium hydroxide or ettringite can readily form. Calcium hydroxide predominates because of its ability to form large crystals, which are often highly oriented approximately normal to the surface.

It is now recognized that the ITZ is an extremely important part of the microstructure, which plays an important role in modifying the properties of concrete. In some cases, it may be a dominant factor. For example, mercury porosimetry of mortars

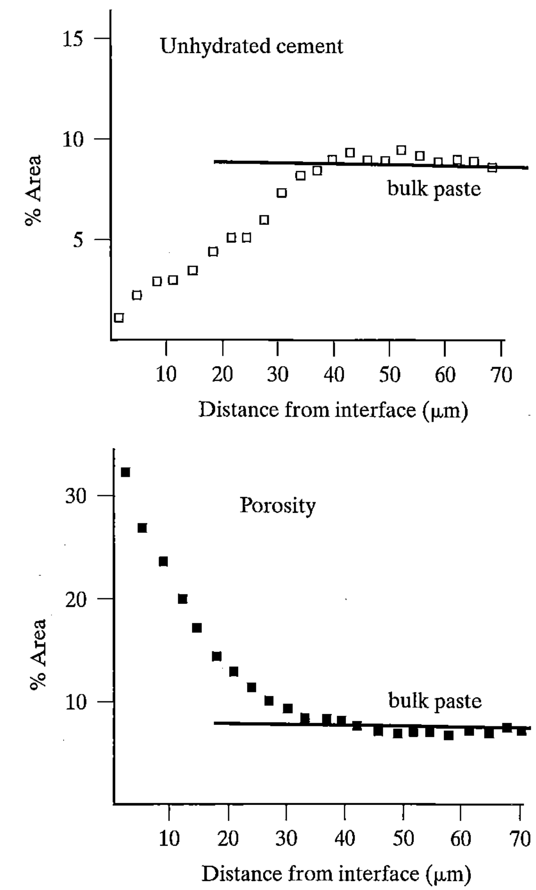


FIGURE 4.12

Variation in volume fraction of selected paste components adjacent to aggregate particles. The ITZ is 30–40 μm.

shows that the interfacial porosity is interconnected macroporosity and that it must therefore strongly influence permeability. The attainment of high strength also, especially in tension, requires the reduction or elimination of the ITZ, which limits the strength of the paste–aggregate bond.

Computer Simulation of Microstructure

Microscopic observations give an excellent qualitative assessment of microstructure. Using modern image analysis, it is possible to determine quantitatively the proportions of each major component, although this is not a trivial exercise. Over the years, mathematical models, largely empirical, have been developed to quantify the effects of hydration and microstructure on properties. These have become more sophisticated as computers have been used for this purpose. In recent years, the computer has been used to simulate the development of microstructure to provide a quantitative description of the complete microstructure to relate to properties. The most successful of these efforts have been the digital image-based models where matter is represented by volume-based pixels, which are manipulated by a series of algorithms describing the hydration processes.

Microscopic images of cement particles dispersed in an epoxy resin are used to model the size distribution, geometry, and arrangement of particles in the paste and the distribution of cement compounds in the particles. Each phase is then simultaneously passed through a series of hydration cycles, each compound obeying its own hydration equation. Each hydration cycle follows the dissolution of the pixels at the exposed surfaces of the cement compounds, their conversion to pixels of hydration products according to the appropriate equations, and their transport to sites of product growth using random walk processes. Hydration can be stopped after any desired cycle. The microstructure is developed in three dimensions and is described mathematically to give complete information about volumes of phases or capillary porosity and their spatial relations to one another. Other operations can then be performed on any component of the microstructure to provide information on properties. Needless to say, such a simulation requires large amounts of memory, and a supercomputer must be used for the calculations.[2]

Graphical depictions of the simulated microstructure match those seen by microscopic image analysis quite well. The model can be used to quantitatively demonstrate the effects on the microstructure of changing the *w/c* ratio and degree of hydration. The simulations can be extended to include model aggregate in the paste, although only mortar can be realistically modeled because of size constraints. Aggregate particles are first placed within the volume, and then the cement particles packed in randomly amongst them. This automatically leads to the creation of the interfacial transition zone.

4.4 PROPERTIES OF HYDRATED CEMENT PASTES

Volume Changes during Hydration

An important aspect of microstructural development is the decrease in porosity during hydration. All the hydration products of the cement compounds have lower specific gravities and larger specific volumes than the cement compounds themselves (see Table 4.8). Thus, every hydration reaction is accompanied by an increase in solid volume.

Expansive Reactions It is not only the potential volume change that is of interest, but also the way in which this change is manifested. In our discussion of microstructure, it was noted that CH either grows around solid particles or stops growing in that particular direction when it meets such obstacles. The same is true of C–S–H. Thus, the hydration of the calcium silicates is not accompanied by an increase in the total volume of the paste. The hydration products will only occupy space that is available to them within the paste, which is the volume originally occupied by the mix water. If this space is filled before complete hydration has occurred, further hydration will cease.

However, bulk expansion occurs when ettringite is formed from aluminate phases. If space is limited, ettringite crystals may develop crystal growth pressures. In a

[2]A simple 2-D educational version, which can run on a PC or Macintosh computer, is available from NIST.

TABLE 4.8 Physical Data Determining Volume Changes That Occur during Hydration

Initial Cement Compounds			*Hydration Products*		
		Molar Volume			*Molar Volume*
Compound	*Specific Gravity*	*(GMW/ρ)[a]* $10^{-6}\,m^3$	*Compound*	*Specific Gravity*	*(GMW/ρ)[a]* $10^{-6}\,m$
C_3S	~3.15	~72.4	C–S–H	2.0[b]	215[b]
C_2S	3.28	52.4	CH	2.24	33.2
C_3A	3.03	89.1	$C_6AS_3H_{32}$	1.78	715
C_4AF	~3.73	~128	C_4ASH_{12}	2.02	313
M	3.58	11.0	MH	2.37	24.2
CSH_2	2.32	74.2	C_4AH_{13}	~2.02	~260
$CSH_{1/2}$	2.74	52.9	C_2AH_8	1.95	165
Portland cement	3.15	–	C_3AH_6	2.52	150

[a]GMW, gram molecular weight; ρ, density in g/m^3.
[b]Value depends on the water content of C–S–H, which is related to how much "gel porosity" is included in the structure. Value given for $C_3S_2H_8$.

paste that has that has not yet hardened, ettringite has plenty of room in which to form, or it can make additional room by pushing aside obstructing cement grains. Once the paste has gained rigidity, however, the extra space necessary for continued formation of ettringite is created by expanding the total volume of the paste.

We have discussed how this bulk expansion is harnessed to good purpose in expansive cements. If not restrained, however, the internally generated expansions will induce internal stresses that may crack and damage the paste. Ordinarily, almost no bulk expansion occurs during the hydration of portland cement pastes through the crystallization of ettringite, because only small amounts form after the paste has hardened. But if too much gypsum is added, then more ettringite will be formed long after setting, which may disrupt the paste sufficiently to lower the compressive strength of the paste or even, in extreme cases, cause cracking. This is the reason for the limitation on SO_3 in ASTM C 150.

Magnesium and calcium hydroxides develop similar crystal-growth pressures when formed directly from their oxides in limited space. Again the pressures will be proportional to the free-energy changes associated with hydration. This phenomenon is quite general and is implicated in the expansive internal growth of ice crystals leading to frost damage in concrete, and in the *in situ* crystallization of salts, which under the right circumstances can lead to deterioration of concrete structures (Chapter 18).

Calculation of Volume Changes

Knowledge of porosity can be very useful since porosity has such a strong influence on paste properties, particularly strength and durability. The total porosity in a paste (but not pore size distributions) can be calculated quite simply. It is possible to calculate volume changes, and hence porosities, from the chemical equations describing hydration using the specific gravities of each species, if the composition and the fraction of

hydration of each compound are known. However, a simple set of equations was proposed by T. C. Powers, which can be applied to all portland cements (Types I to V) and which avoids the more rigorous and complex calculations. These equations are empirical, being derived from experimental data. In the form given here, the equations can be easily solved if the amount of hydration of the cement is known. The volume relationships among the constituents are presented schematically in Figure 4.13, ignoring the realities of the microstructure for simplicity. The hydrated cement fraction includes all the hydration products, C–S–H, CH, and sulfoaluminates, without distinguishing them separately.[3]

Two types of water are distinguished: *Evaporable water* is lost when a saturated paste is D-dried (or oven-dried) and *nonevaporable water* is lost when a D-dried paste is heated (ignited) to 1000°C (1800°F). Evaporable water encompasses the water held in both capillary and gel pores (including interlayer pores) as well as some hydrate water from the calcium sulfoaluminates. Nonevaporable water approximately measures the amount of water combined structurally in the hydration products. The nonevaporable water (w_n) is proportional to the amount of hydration that has occurred, so that

$$w_n = 0.24\alpha \text{ g/g of original cement} \qquad (4.10)$$

where α is the degree of hydration (i.e., the fraction of cement that has hydrated). When the cement is fully hydrated ($\alpha = 1.0$), 0.24 g of nonevaporable water is combined with each gram of cement. Equation (4. 10) is used to determine α experimentally. The amount of evaporable water associated with all the hydrated components[4] (which includes some hydrate water from the calcium sulfoaluminates) is given by the relationship

$$w_g = 0.18\alpha \text{ g/g of original cement} \qquad (4.11)$$

where w_g is the *gel water* and is primarily associated with the C–S–H.

The total volume of hydration products (cement gel) is given by

$$V_{hp} = 0.68\alpha \text{ cm}^3\text{/g of original cement}^5 \qquad (4.12)$$

The volume of gel pores (V_g) is also determined by Eq. (4.11) as cm³/g of original cement, since the specific gravity of evaporable water is effectively 1.0. Gel porosity (P_g), expressed as a fraction of V_{hp}, is

$$P_g = \frac{w_g}{V_{hp}} = 0.26 \qquad (4.13)$$

This fraction is a constant value for all normally hydrated cements. Thus, about one-fourth of the volume of the hydration products is pore volume.

[3]Powers called the hydrated fraction "cement gel." The individual fraction of each constituent can be calculated if the hydration equations are used.
[4]This water is held in the intrinsic porosity of the cement gel (i.e., the gel pores).
[5]The units used here are those originally used; the proper SI unit would be m²/g.

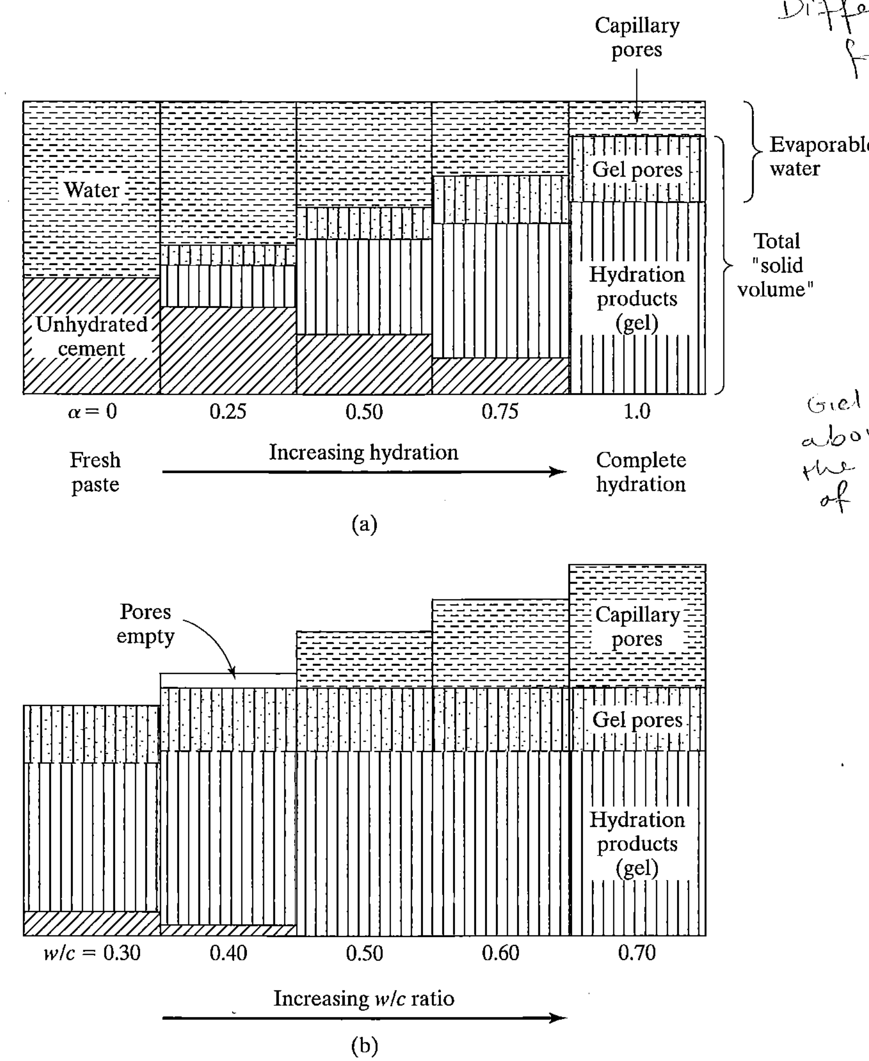


FIGURE 4.13

Volume relationships among constituents of hydrated cement pastes: (a) constant *w/c* ratio = 0.5; (b) changing *w/c* ratio ($\alpha = 1.0$).

The capillary pore volume is given by

$$V_c = \frac{w}{c} - 0.36\alpha \quad \text{cm}^3\text{/g of original cement} \qquad (4.14)$$

where *w* is the original weight of water used to form the paste. *w/c* is therefore the familiar water/cement ratio (by weight) and capillary porosity is strongly dependent on the *w/c* ratio.

The volume occupied by the unhydrated cement is

$$V_u = (1 - \alpha)v_c \text{ cm}^3\text{/g of original cement} \qquad (4.15)$$

where v_c is the specific volume (reciprocal of specific gravity) of cement and is approximately 0.32 (see Table 4.7). Before hydration has commenced, V_u is simply equal to v_c. Therefore, the original volume of the paste is

$$V_p = w/c + v_c \text{ cm}^3/\text{g of original cement} \tag{4.16}$$

As the w/c ratio of a paste is increased, its capillary porosity increases. The capillary porosity is

$$P_c = V_c/V_p = \frac{w/c - 0.36\alpha}{w/c + 0.32} \tag{4.17}$$

Powers also used the gel/space ratio, which is a measure of capillary porosity and is defined as

$$X = \frac{\text{volume of gel (including gel pores)}}{\text{volume of gel + volume of capillary pores}} = \frac{0.68\alpha}{0.32\alpha + w/c} \tag{4.18}$$

Using Eqs. (4.10) through (4.18), it is possible to calculate the volume of paste constituents for varying values of α and w/c ratios. This was done to construct Figure 4.13. These calculations assume that no overall expansion occurs and that pastes are hydrated at, or near, ambient temperature. It is also assumed that the pastes are hydrated in a sealed condition; that is, with no additional water added during curing and none lost by evaporation. Figure 4.13 clearly shows the marked effects that w/c and α have on capillary porosity, as indicated by Eq. (4.14).

A sample calculation for $w/c = 0.5$ and $\alpha = 0.75$ is

$$w_n = 0.24 \times 0.75 = 0.18 \text{ g/g of original cement}$$

$$w_g = 0.14 \text{ g/g original cement}$$

$$V_g = 0.14 \text{ cm}^3/\text{g original cement}$$

$$V_{hp} = 0.68 \times 0.75 = 0.51 \text{ cm}^3/\text{g original cement}$$

$$V_c = 0.50 - (0.36 \times 0.75) = 0.33 \text{ cm}^3/\text{g original cement}$$

$$V_u = 0.25 \times 0.32 = 0.08 \text{ cm}^3/\text{g original cement}$$

$$P_g = 0.26 \text{ (independent of } \alpha)$$

$$P_c = 0.33/(0.50 + 0.32) = 0.40$$

$$X = 0.51/[(0.32 \times 0.75) + 0.50] = 0.69$$

Minimum *w/c* Ratios It can be seen that at low w/c ratios, there is insufficient space for the hydration products to form so that complete hydration is not possible. Using this criterion, the minimum w/c ratio that can be used and still ensure complete hydration can be determined from Eq. (4.14) by setting $P_c = 0$ and $\alpha = 1.0$; the value is 0.36. However, the hydration products must be formed with the gel pores saturated. Thus, the water required for complete hydration is

$$w_{min} = (w_n + w_g) \text{ g/g of original cement} \tag{4.19a}$$

or

$$(w/c)_{min} = 0.42\alpha \tag{4.19b}$$

Thus, for complete hydration ($\alpha = 1$), the w/c ratio should not fall below 0.42.

Self-Desiccation The space requirements for gel are less than the water requirements, so that the available water will be used up while space is still available. This means that below a w/c ratio of 0.42, a paste will *self-desiccate* and the residual capillary pores will become partially empty, unless water is added during the curing period. Since water is physically lost from the paste by evaporation, absorption by formwork or subgrade, etc. during actual concreting, the effective minimum w/c ratio needed to avoid self-desiccation is higher than 0.42.

Unhydrated cement will therefore remain in pastes made with very low w/c ratios, as used in high-strength concrete. Clearly, complete hydration is not essential to attain a high ultimate strength. In fact, the contrary is true: A high proportion of unhydrated cement will enhance strength when capillary porosities are low. With regular concrete, self-desiccation is seldom a problem in the field, since there are more severe consequences of inadequate moist curing that will overshadow any other effects. However, self-desiccation can become important in pastes with low w/c ratios (particularly those containing silica fume) even when moist cured, because water cannot easily penetrate into the dense microstructure. The internal relative humidity can drop to 75–80%, leading to bulk shrinkage, which is called *autogenous shrinkage* (Figure 4.14). Restraint of this shrinkage can lead to internal microcracking, but the effect of this on concrete performance has not been fully assessed.

Influence of Hydration Products on Paste Properties

It will now be appreciated that hardened cement paste is composed of a complex mixture of diverse components. On the microscopic level, the paste must definitely be considered as an inhomogeneous, anisotropic, composite matrix. Yet from a macroscopic viewpoint, the assumption of a homogeneous, isotropic material has validity since the hydration products are closely intergrown and well dispersed throughout the paste. The properties of the paste components must be considered when attempting to understand and predict the properties of the hydrated paste. A general consideration of the influence of the paste composition will be given here, although the subject will be discussed further in later chapters. Since C–S–H makes up the bulk of the paste matrix, it is reasonable to assume that it has a dominant effect on paste properties. This is indeed true, the question being to what extent the other components modify the properties of the paste.

Chemical Bonding The nature of chemical bonding within a material determines the mechanical response of that material to different states of stress and the relationships between the various parameters describing mechanical response. Previously, it was thought that the very low tensile strength of the paste was a result of van der Waals'

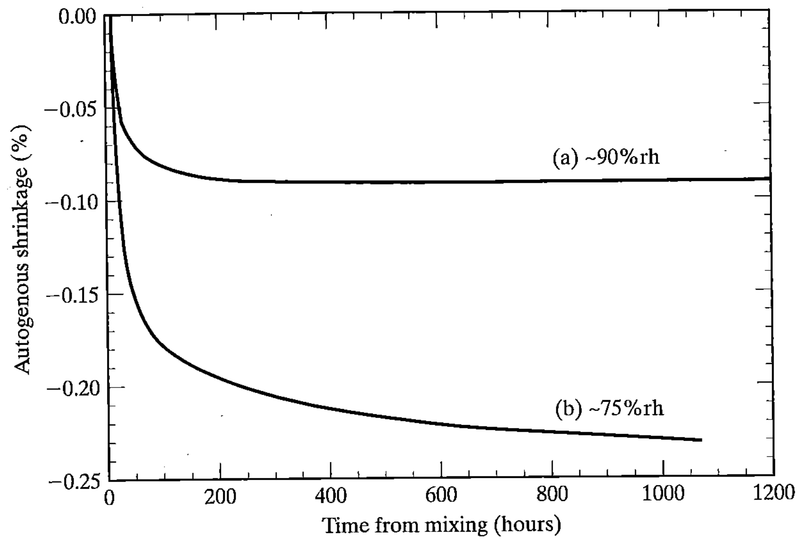


FIGURE 4.14

Autogenous shrinkage of cement paste (w/c ratio = 0.2) with the approximate internal RH shown: (a) plain paste; (b) with added silica fume. [From H. Ai, Ph.D. Thesis, University of Illinois at Urbana–Champaign (2000).]

bonding between the surfaces of the colloidal particles of C–S–H. Now it is recognized that hardened cement paste more closely resembles ceramic materials (dominated by ionic-covalent bonding in their mechanical behavior) than it does organic polymers (dominated by van der Waals' bonding). The low tensile strength of concrete is rather the consequence of the presence of flaws on a much larger scale than atomic bonds (see Chapter 13). Nevertheless, there is probably an appreciable van der Waals' component, which may perhaps be one-third to one-fourth of the total bonding energy. The influence of van der Waals' bonding is reflected in the unique creep and shrinkage behavior of concrete and the influence of moisture on these properties (and only to a lesser extent on mechanical properties).

Porosity and Pore Structure

Gel/Space Ratio. In their classic work published in 1946–47, Powers and Brownyard showed that the increase in compressive strength of portland cement mortars "is directly proportional to the increase in gel/space ratio regardless of age, original w/c ratio, or identity of cement." The gel/space ratio [Eq. (4.18)] is the ratio of the volume of solid products of hydration to the space available for these hydration products. In other words, the gel/space ratio is a representation of the capillary porosity of the paste expressed in terms of α and w/c. Before hydration begins ($\alpha = 0$), the available space is that occupied by the mixing water. After hydration has proceeded to a certain degree, the available space is then the sum of the volumes of cement that has been hydrated and the original water. (Remember that 1 cm^3 of cement will occupy about 2.1 cm^3 of space when fully hydrated.)

The relationship between compressive strength (σ_c') and the gel/space ratio can be written as

$$\sigma_c' = AX^n \tag{4.20}$$

where A is a constant representing the intrinsic strength of the cement gel (i.e., the strength at $X = 1.0$) and n is a constant having values in the range 2.6 to 3.0, depending on the characteristics of the cement. The relationship found by Powers and Brownyard (Figure 4.15) can be written as

$$\sigma_c' = \begin{cases} 235\,X^3 & \text{MPa} \\ 34{,}000\,X^3 & \text{lb/in.}^2 \end{cases} \tag{4.21}$$

This expression is independent of the mix design or age. It should be noted, however, that these data are based on strengths obtained from 50 mm (2 in.) mortar cubes. Tests on neat cement paste, or on specimens of different geometries, would presumably lead to different values of the constants, although the general trend would be the same. Furthermore, the gel/space ratio concept is not entirely independent of the cement composition; higher C_3A contents (particularly above 7% C_3A) lead to lower strengths at a given value of X.

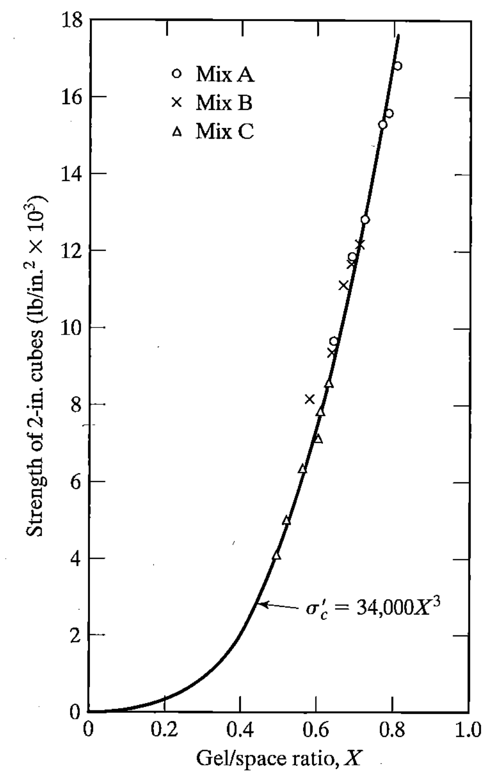


FIGURE 4.15

Compressive strength vs. gel/space ratio for cement-sand mortars; σ_c' = compressive strength (lb/in.^{2}); X = gel/space ratio. [From T. C. Powers, *Journal of the American Ceramic Society*, Vol. 41, No. 1, pp. 1–6 (1958).]

In the preceding discussion, the gel porosity is included as a part of the solid C–S–H. This accounts for the relatively low intrinsic strength (235 MPa or 34,000 lb/in.^{2}) of the paste, since C–S–H, which is a major component of the hardened paste, is a highly porous solid. It is possible to express the strength as a function of the *total* porosity (i.e., capillary + gel porosity), but there is no practical advantage in doing so because the gel porosity is a constant fraction of the C–S–H volume.

Transport properties. As we have just discussed, the strength of cement paste depends primarily on the porosity, a property that it holds in common with other porous materials; the exact chemical nature of the hydration products is less significant. As long as there is enough water present (theoretically) to fully hydrate the cement, the gel porosity is independent of the *w/c* ratio [Eq. (4.13)]; it depends only on the degree of hydration, or maturity, of the paste. Thus, as may be seen from Figure 4.13, the addition of water in the normal range of *w/c* ratios for concrete (0.40 to 0.55) affects only the capillary porosity. Since the gel porosity is an intrinsic part of the paste structure, this means that changes in many concrete properties will be directly related to changes in the capillary porosity. Different pore sizes affect different properties, as shown in Table 4.7. Very large pores, or areas of high local porosity, can act as crack initiators. Macropores control the transport of water or dissolved ions through the paste and either the volume intruded by mercury above 0.1 μm or the continuous pore diameter correlates well with permeability. As will be discussed in Chapter 18, well-hydrated pastes with low *w/c* ratios have permeabilities that may be more than three orders of magnitude lower than a paste with a high *w/c* ratio. This is because the larger pores become isolated, and water movement is controlled by flow through the gel pores, through which it proceeds only with difficulty. Computer simulations show that when the total capillary porosity falls below 20% the pores become disconnected, as shown in Figure 4.16.

Drying shrinkage. Drying shrinkage is controlled by the mesopore system and is also influenced by concomitant changes taking place in the micropores, while creep is controlled primarily by the micropores. These phenomena are discussed more fully in Chapter 16.

Role of Calcium Hydroxide Calcium hydroxide contributes to strength since it reduces pore volume. Any solid material of comparable stiffness that reduces porosity must increase strength regardless of its chemical composition. However, it has a detrimental effect on durability since it is more soluble than C–S–H. Leaching of calcium hydroxide provides a point of entry for aggressive agents by increasing permeability and diffusivity, as well as weakening the matrix. The alkaline-buffering capacity of calcium hydroxide will slow down acid attack and protect reinforcing steel from corrosion by passivating the surface, but may contribute to alkali-aggregate attack.

Role of Calcium Sulfoaluminates The calcium sulfoaluminate hydrates also affect the properties of the paste. We have discussed sulfate attack, the most dramatic effect,

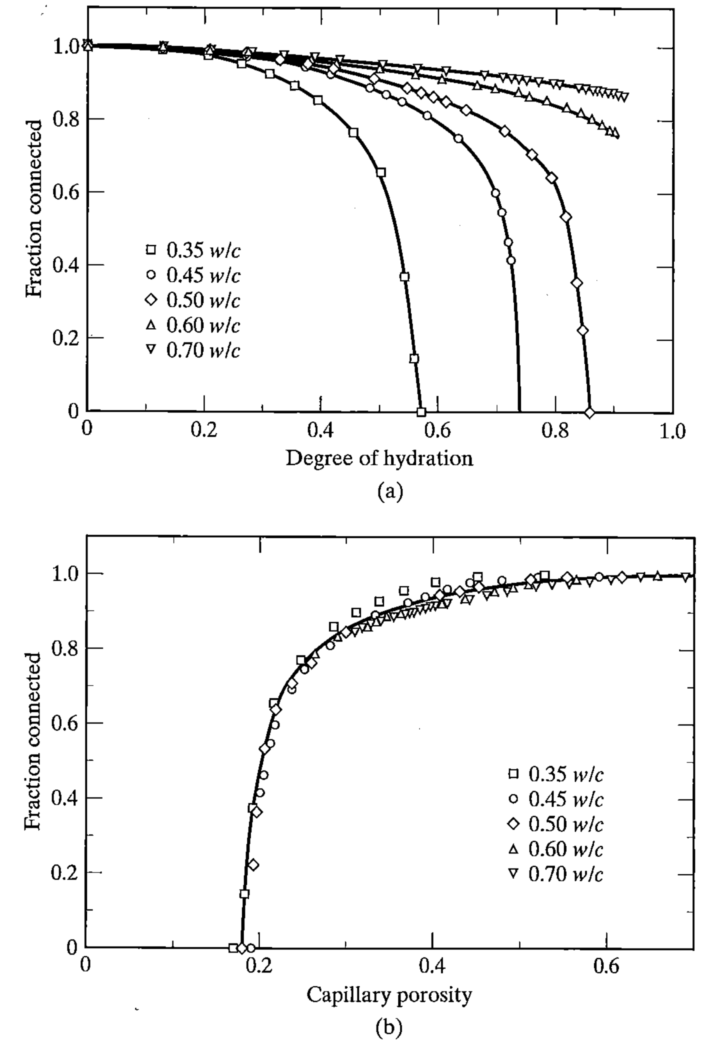


FIGURE 4.16
(a) Effect of hydration and *w/c* ratio on the connectivity of the capillary pore system. (b) These curves collapse to a single curve when plotted against porosity. (Data from computer simulation modeling by E. J. Garboczi.)

and other consequences of the expansive formation of ettringite. The formation of ettringite or monosulfoaluminate can affect the rheology of fresh concrete. Although ettringite can contribute to early strength, excessive formation can cause a subsequent decrease.

Role of Water Water is a polar molecule; therefore, strong secondary interactions (e.g., hydrogen bonding) with hydroxylated surfaces in hydrated paste are to be expected. Because of its highly polar nature and small molecular size, water appears to

play a special role in the structure of C–S–H, as discussed earlier, and hence has an important influence on concrete properties. Saturated concrete is about 20% weaker in compression than is dry concrete. This can be due to three reasons: (1) As adsorbed water is removed, C–S–H particles can come closer together and produce a stronger system due to an increase in van der Waals' bonding; (2) water may attack Si-O-Si bonds under stress; and (3) water may reduce mechanical interlock by acting as a lubricant.

The presence of water in C–S–H is also important in creep and shrinkage, as discussed in Chapter 16. The high degree of hydrogen bonding within water results in high surface tension, which is responsible for the development of large capillary stresses. The movement of water in micropores is a critical factor in these volume changes. The strong affinity of water for surfaces must also be important in admixture chemistry, as chemical admixtures must themselves be highly polar molecules.

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PROBLEMS

4.1. Discuss the effect of temperature on the hydration of portland cement.

4.2. Describe the chemical reactions involved in the sulfate attack of hardened cement paste.

4.3. What are the roles of C_3A and C_4AF in cement hydration?

4.4. Discuss the role of gypsum in the hydration of portland cement.

4.5. Compare the properties of C–S–H and CH, and their roles in determining the properties of hardened cement paste.

4.6. What would you expect the engineering properties of a very high specific surface area (100–400 m^2/g) material like C–S–H to be?

4.7. Discuss the problems involved in measuring the porosity of cement paste.

4.8. Describe the classification of porosity in hardened cement paste.

Special topics exam

4.9. Calculate (a) the volume of hydration products; (b) the capillary porosity; and (c) the gel/space ratio, given the following experimental data: $\alpha = 0.80$, $w/c = 0.45$.

degree of hydration

4.10. Calculate the same quantities as in Problem 4.9 given $w_n = 0.14$ and $w/c = 0.42$.

4.11. What happens in Problem 4.9 if *w/c* is lowered to 0.27?

4.12. Calculate the same quantities in Problem 4.9 for *w/c* = 0.30, 0.40, 0.50, and 0.60.

CHAPTER 5

Mineral Admixtures and Blended Cements

The addition of finely ground solid materials to concrete is an established practice in modern concrete technology. Collectively, these materials are called *mineral admixtures* or *supplementary cementing materials*. Admixtures are added for a variety of purposes with the mineral components being used primarily to replace cement, to improve the workability of fresh concrete, or to enhance the durability of hardened concrete. (Water-soluble additions are called chemical admixtures and will be discussed in Chapter 8.) Mineral admixtures can be divided into three main categories:

1. Pozzolanic materials
2. Cementitious materials
3. Non-reactive materials

In Europe, commercial cements are always blends of portland cement with mineral admixtures, but in the United States such blended cements are less common. It is more usual for the specification to require the concrete producer to add a material from Categories 1 and 2 at the mixer as a *supplementary cementing material*. In either case, the end result is the same: The mineral admixture interacts chemically with the hydrating portland cement to form a modified paste microstructure.

The use of mineral admixtures can be advantageous in several ways. Technical advantages, such as improved durability, will be discussed at length; but many ready-mix companies use fly ash simply on economic grounds, since it is cheaper than cement. (Slag is comparable in price, while silica fume is much more expensive). Mineral admixtures are environmentally friendly not only because waste products are turned into valuable commodities, but also because the energy consumption and CO_2 emissions associated with the production of concrete are reduced.

Materials in Category 3 encompass finely divided materials such as ground limestone, silica flour, hydrated lime, etc. Although classed as nonreactive, some may react weakly with the cement under certain conditions. These substances are blended with portland cement to form masonry cements with improved workability and water

retention. Since improvement in workability is the main use of these fine substances, they are also referred to as *workability aids*. Color pigments also are included in this category.

In this chapter, we will confine our discussion to pozzolanic and cementitious materials, first considering the different classes of materials and then the use of blended cements.

5.1 POZZOLANIC MATERIALS

History and Origins

The addition of naturally occurring materials of volcanic origin to hydraulic limes was known to the Greeks, who used a volcanic ash from the island of Santorin (Santorin earth). (This same material was used both in the Parthenon and the Suez Canal.) The Romans adopted and extended the Greek technology using ash from a variety of sources from around their empire. The name *pozzolan* is a corruption of Pozzuoli, a town in the Bay of Naples that was the source of a highly prized deposit of weathered ash (tuff) from Mt. Vesuvius. (The mortar used in Smeaton's Eddystone lighthouse contained pozzolan from near the modern city of Civitavecchia.) The name is now applied to any reactive aluminosilicate material, of either natural or industrial origin (Table 5.1). The materials must either occur in a finely divided state or be ground to cement fineness.

Natural Pozzolans Naturally occurring pozzolans were used extensively in the construction of mass concrete for hydraulic structures in the western United States during the 1930s and 1940s. However, the first modern use of a pozzolan in the U.S. dates back to the early construction of the Los Angeles aqueduct in 1910–12, and pozzolans also were used in the Golden Gate and San Francisco–Oakland bridges; these projects required sulfate resistance, as well as reduced heat of hydration. The Bonneville dam in 1935 was the first of many mass concrete dams to use natural pozzolans to control the temperature rise during hardening. Local materials were used for this purpose, mostly

TABLE 5.1 Common Pozzolanic Materials

Category	*Typical Materials*	*Active Components*
Natural Materials	unaltered volcanic ash	aluminosilicate glass
	weathered volcanic ash (tuff, trass, etc.)	aluminosilicate glass; zeolites
	pumice	aluminosilicate glass
	diatomaceous earth	amorphous hydrated silica
	opaline cherts and shales	hydrated silica gel
By-product Materials	fly ash - Class F	aluminosilicate glass
	- Class C	calcium aluminosilicate glass
	silica fume	amorphous silica
	rice husk ash	amorphous silica
	calcined clays	amorphous alumino-silicate (metakaolin)

of volcanic origin, but some diatomaceous earths and opaline materials were used. Some of these materials required calcining to enhance their pozzolanicity by activating clay mineral impurities. Calcined clay was used by the Romans in the form of pulverized broken tile in pottery as a substitute for volcanic ash, and this material has now returned to the construction site in the form of metakaolin.

By-Product Materials The pozzolans in use today are commonly by-product materials that are widely available. *Fly ash*, which is the most extensively used material, is the inorganic, noncombustible residue of powdered coal after burning in power plants. The molten particles are swept out of the furnace with the stack gases and collected by electrostatic precipitators after cooling. Fly ashes are employed in a wide variety of construction, and many ready-mix companies add fly ash routinely to their concrete. Although their motivation is mainly economic (fly ash is less than one-half the price of cement), the addition of fly ash has many technical benefits, discussed more fully later. Over 250 million tons of fly ash are produced each year in the U.S., far more than can be used in concrete. However, some fly ashes are not suitable for use in concrete for various reasons, particularly some that derive from lignitic coals from western states. As power companies turn to scrubber systems to remove sulfur dioxide from stack gasses, many fly ashes become mixed with scrubber products and contain free lime and calcium sulfates or sulfites. These materials cannot be used in concrete. It is advisable to purchase fly ash from a reputable supplier to ensure predictable and reliable performance.

Silica fume is a by-product in the manufacture of silicon metal and alloys. The material is formed by vapor phase oxidation in the stack gases and collected in baghouses. About 200,000 tons of silica fume are produced each year in the U.S. *Rice husk ash* is the siliceous residue that remains after rice husks are burnt under controlled conditions to remove the organic matter. Rice husks contain about 20 wt.% of silica, much higher than any other crop residue. It is estimated that about 90 million tons of rice husks are produced worldwide each year, but less than 2% of this is produced in the U.S. Many clays can be calcined under controlled temperatures (700–900°C) to produce a highly reactive amorphous aluminosilicate (metakaolin).

Pozzolanic Reactions

The amorphous or glassy silica, which is the major component of a pozzolan, reacts with calcium hydroxide formed from the hydration of the calcium silicates. The principal reaction is

$$\text{CH} + \text{S} + \text{H} \rightarrow \text{C–S–H} \tag{5.1}$$

When fly ash and natural pozzolans are used, the composition of C–S–H is not very different from that formed in regular hydration, although generally the C/S molar ratio is slightly lower. However, with very reactive pozzolans of high silica content (silica fume and rice husk ash), the C/S ratio is significantly different, being close to 1.0, and the H/S ratio is slightly lower. This indicates that a secondary pozzolanic reaction with C–S–H is occurring. Thus, the overall reaction can be written as

$$C_3S + 2S + 10.5H \rightarrow 3[CSH_{3.5}] \tag{5.2}$$

Small quantities of reactive alumina in a pozzolan generally substitute for silica as part of the C–S–H. When a pozzolan has appreciable quantities of reactive alumina (e.g., natural pozzolans or calcined clay), a separate set of secondary reactions can occur, leading to the formation of calcium aluminate hydrates C–A–H:

$$CH + A + H \rightarrow C\text{–}A\text{–}H \tag{5.3}$$

The exact composition of the calcium aluminate hydrates depends on the particular pozzolan; AFm compounds such as C_2AH_8, C_2ASH_8 (gehlenite hydrate), or monosulfoaluminate may form, depending on the composition of the pozzolan.

Equation (5.1) is the primary pozzolanic reaction, and its kinetics are more similar to the slow rate of hydration of C_2S. The heat of reaction is about −12 kJ/mole of CH compared to −43 kJ/mole of C_2S. Thus, the addition of a pozzolan has a similar effect to raising the C_2S content of the cement, thereby lowering the amount of early heat evolution and reducing early strength, but not long-term strength. The activation energy of the pozzolanic reaction is about 30 kJ/mol. Since the pozzolanic reactions have an overall increase in solid volume, the porosity of the paste will eventually be reduced, resulting in higher strength and durability compared with a plain paste of comparable reaction.

The extent of a pozzolanic reaction can be followed by monitoring the decrease in calcium hydroxide over time. Results of such an analysis are shown schematically in Figure 5.1, which shows the high reactivity of silica fume and calcined kaolin. In theory, it is possible to completely react all the calcium hydroxide formed by regular hydration, but this seldom happens in practice. In fact, it is important to realize that the slow rate of the pozzolanic reaction requires a prolonged period of moist curing if the full benefits of adding a pozzolan are to be realized. Without sufficient moist curing, a pozzolan will act mainly as a noncementitious filler. The pozzolanic reaction is more temperature sensitive than regular hydration and can proceed quite rapidly under steam curing.

Pozzolanic capacity is usually tested by direct reaction with $Ca(OH)_2$ and monitoring the consumption of lime (or strength development). Comparisons are given in Table 5.2, which shows that Class C fly ash has a lower capacity than Class F because of

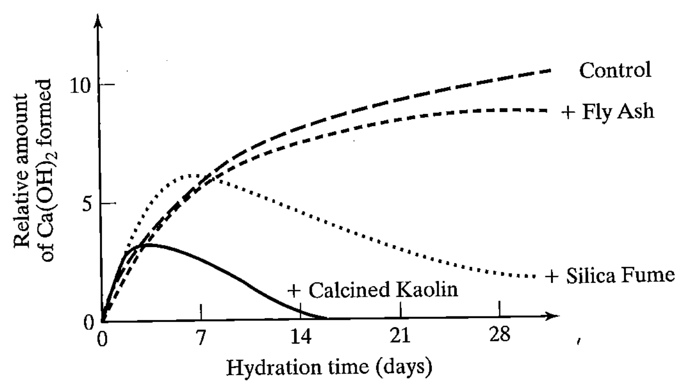


FIGURE 5.1

Comparison of the rates of reaction for various pozzolanic admixtures.

TABLE 5.2 Typical Pozzolanic Capacity of Selected Pozzolans

Pozzolan	*mg $Ca(OH)_2$ consumed per g*
Blast Furnace Slag	40
Silica Fume	400
Fly Ash - Class C	500
Class F	850
Metakaolin	1000

its high lime content. Higher alumina content of a pozzolan means higher capacity because formation of C–A–H has a high lime demand.

Composition

Chemical Analysis Because pozzolans are such diverse materials, it would be expected that their overall chemical composition would be quite variable, and this is indeed the case, as shown in Table 5.3. For this reason, ASTM specification C 618 for fly ash and natural pozzolans is primarily a performance specification, although there is a chemical requirement that the sum of the major acidic oxides (SiO_2 + Al_2O_3 + Fe_2O_3) must not be less than 70% (50% for Class C fly ash). However, this requirement unrealistically assumes that all of these oxides are able to participate in the pozzolanic reaction. A major provision of the ASTM specification is a minimum strength activity index of 75% at 7 or 28 days, as determined by ASTM C 311. The index compares the cube compressive strength of mortar containing a 20% pozzolan replacement of cement with that of a control mix containing no pozzolans. Other provisions relate to other aspects of engineering performance such as sulfate resistance.

Fly ashes are subdivided into two classes, F and C, which reflect the composition of the inorganic fractions. Class F fly ashes are produced from bituminous and subbituminous coals that are found in states east of the Mississippi river. Class C fly ashes derive from the lignitic coals of western states, principally Wyoming and Montana. Class C fly ashes contain high levels of calcium oxide, much of which is contained in the glassy fraction.

TABLE 5.3 Typical Bulk Analyses of Pozzolans (weight %)

Material	*SiO_2*	*Al_2O_3*	*Fe_2O_3*	*CaO*	*Na_2O*	*K_2O*	*Carbon*
Volcanic Glass	50–70	15–20	1–6	2–10	2–4	2–10	—
Volcanic Tuffs	45–75	10–20	1–10	1–10	<2	2–7	—
Diatomaceous Earth	~85	~2	~2	<1	—	—	
Fly Ash - Class F	>50	20–30	<20	<5	Variable		<5
- Class C	>30	15–25	<10	20–30	Variable		<1
Silica Fume	85–98	<2	<10	—	Variable, low		<2
Rice Husk Ash	85	—	—	<1	1–4	3–10	3–18
Calcined clay	~55	35–45	<10	—	<1	<1	—

Crystalline Components An overall chemical analysis, while helpful, does not adequately characterize a pozzolan. A good pozzolan should have a high fraction of reactive glassy[1] or amorphous material. This ideal is approached in a good-quality silica fume, with up to 98% of amorphous silica, while fly ash and natural pozzolans generally contain significant quantities of inert crystalline phases. Crystalline impurities can be identified by x-ray diffraction; typical patterns are shown in Figure 5.2. The amorphous or glassy fraction produces a characteristic "hump" whose position varies according to its CaO content. The sharp peaks superimposed on the hump are used to identify crystalline phases. Specialist analytical laboratories can provide the information about glass contents and crystalline phases.

Good-quality Class F fly ashes have 70–90% glass, with quartz, mullite, hematite, and magnetite as the principal phases that crystallize from the glass on cooling. Almost all of the iron is contained in the latter two phases. Class C fly ashes contain quartz, free lime (uncombined CaO), and anhydrite ($CaSO_4$). Some of these fly ashes also may contain $C_4A_3\bar{S}$, C_3A, and C_2S; these fly ashes will show cementitious tendencies and will become hard on the addition of water, whereas other pozzolans do not react with water alone. In general, cementitious Class C fly ashes should not be used, to avoid problems with high water demand, early stiffening, or rapid setting.

Natural pozzolans have more variable glass contents, ranging from 50 to 90%. The principal crystalline compounds are quartz and feldspars, but some (e.g., volcanic tuffs) contain zeolites, which although crystalline can act as pozzolans.

Minor Components There are a few other concerns regarding composition. Many pozzolans contain alkali oxides (alkalis). In the case of fly ash, these are often in the form of soluble salts that have been deliberately added to improve the efficiency of the electrostatic precipitators, and they may occasionally influence the rate of setting. In the case of natural pozzolans, the alkalis are usually in an insoluble form and are released only during reactions, if at all. Since pozzolans are added to control alkali-aggregate reactions, the addition of more alkali is counterproductive. If a pozzolan is to be used for this purpose, the alkali content should be determined, so that its contribution to the total alkalis in the concrete can be calculated.

Many of the by-product materials contain unburned carbon: 1–2% for silica fume, 0.5–3.0% for fly ashes (but sometimes as high as 25%), and 3–18% for rice husk ash. The high carbon content in some fly ashes can be reduced to a low level using electrostatic separators to make them more suitable for use in concrete. (The high level of carbon in many rice husk ashes has inhibited the large-scale commercial use of this material despite its high pozzolanic activity. Specially designed furnaces must be used to completely remove residual carbon from decomposing cellulose at a low enough temperature so that the amorphous silica will not crystallize.) Carbon contents are estimated from loss on ignition. The main concern about residual carbon is its detrimental effects on air entrainment. Higher doses of air-entraining agents will be required, and in extreme cases satisfactory air entrainment may not be possible. Carbon also affects the color of the ash, although iron contents are also important in this regard; hence, pozzolans may range in color from very light off-white shades to almost black. ASTM

[1] Glassy material is formed by rapid cooling of a high-temperature liquid, which inhibits crystallization.

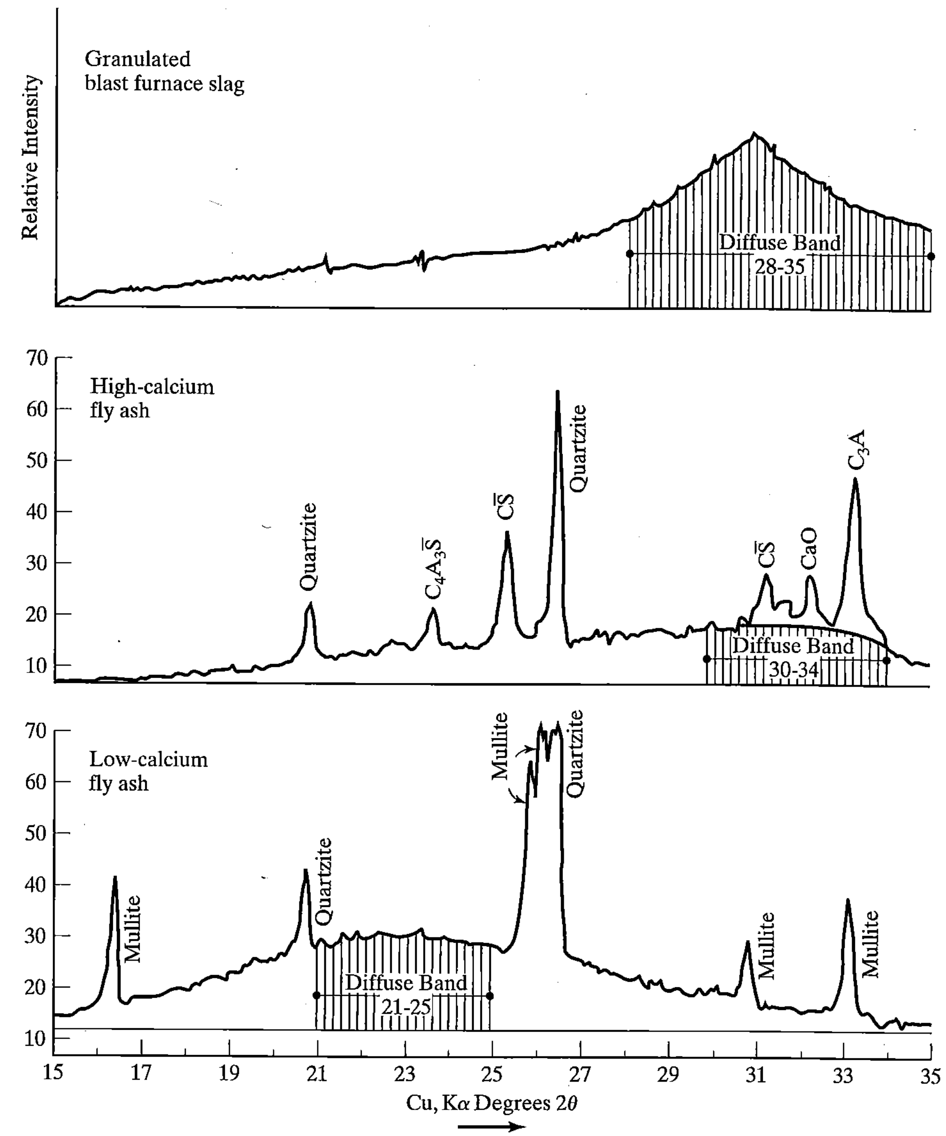


FIGURE 5.2

X-ray diffraction patterns of fly ash and granulated blast-furnace slag.

C 618 sets limits for carbon through loss of ignition, and there are optional limits on the amount of alkalis. It also restricts the amount of SO_3 and reduces the chance for unsoundness due to the hydration of periclase through the autoclave expansion test.

Physical Characteristics Pozzolans must be in a finely divided state to be effective. Most natural materials require grinding to cement fineness, but the industrial by-products often exist as fine particles and need not be ground. A comparison of some materials is given in Table 5.4, where it can be seen that particle morphology is highly variable. Spherical particles are highly desirable because they have a positive impact on workability, while cellular and platey particles impart a higher water demand. Typical morphologies of fly ash and silica fume are shown in Figure 5.3. ASTM C 618 limits the amount of material retained on the 45 μm (No. 325) sieve, but this is really an inadequate assessment of particle size and shape. A more useful parameter is the surface area, since the rate of pozzolanic reaction will be proportional to the amount of surface available for reaction. The high surface area of rice husk ash in relation to its mean particle size is a consequence of its cellular nature and is the reason for its high reactivity, but also results in a high water demand.

A complete particle-size distribution analysis is even more useful and comparisons are shown in Figure 5.4. The extremely fine nature of silica fume is very evident and is the reason for its high activity. However, such a fine material is very difficult to handle, so that silica fume is generally supplied in more easily handled forms, either as a slurry in water, or more commonly in a densified state. The latter material is mechanically agglomerated, and the agglomerates should be redispersed in the concrete mixer. If nearly complete dispersion is not achieved, much of the benefits of adding silica fume will be lost. However, it is not unusual to find some large agglomerated particles that have not been dispersed. One of the benefits of the very fine nature of silica fume is its ability to pack between the cement particles. This *particle packing* characteristic sets it apart from other pozzolans, although it can be achieved to some extent with calcined clays. Particle packing reduces bleeding, lowers the mean size of the capillary pores, and can reduce water requirements. Between 5 and 10% (by mass of cement) is required to provide significant effects, and a water-reducing agent should be used to disperse both silica fume and cement. With this combination, *w/cm* (water to cementitious materials) ratios below 0.25 can be achieved while maintaining good flowability. Very fine fly ash fractions < 5 μm obtained by air classification can also provide some particle packing.

TABLE 5.4 Physical Characteristics of Selected Pozzolans

Material	Mean Size (μm)	Surface Area (m^2/g)	Particle Shape Gravity	Specific Gravity
Portland Cement	10–15	<1	angular, irregular	~3.2
Natural Pozzolans	10–15**	<1	angular, irregular	variable
Fly Ash (F and C)	10–15	1–2	mostly spherical	2.2–2.4
Silica Fume	0.1–0.3	15–25	spherical	2.2
Rice Husk Ash	10–20**	50–100	cellular, irregular	<2.0
Calcined Clay (metakaolin)	1–2	~15	platey	2.4

**After grinding.

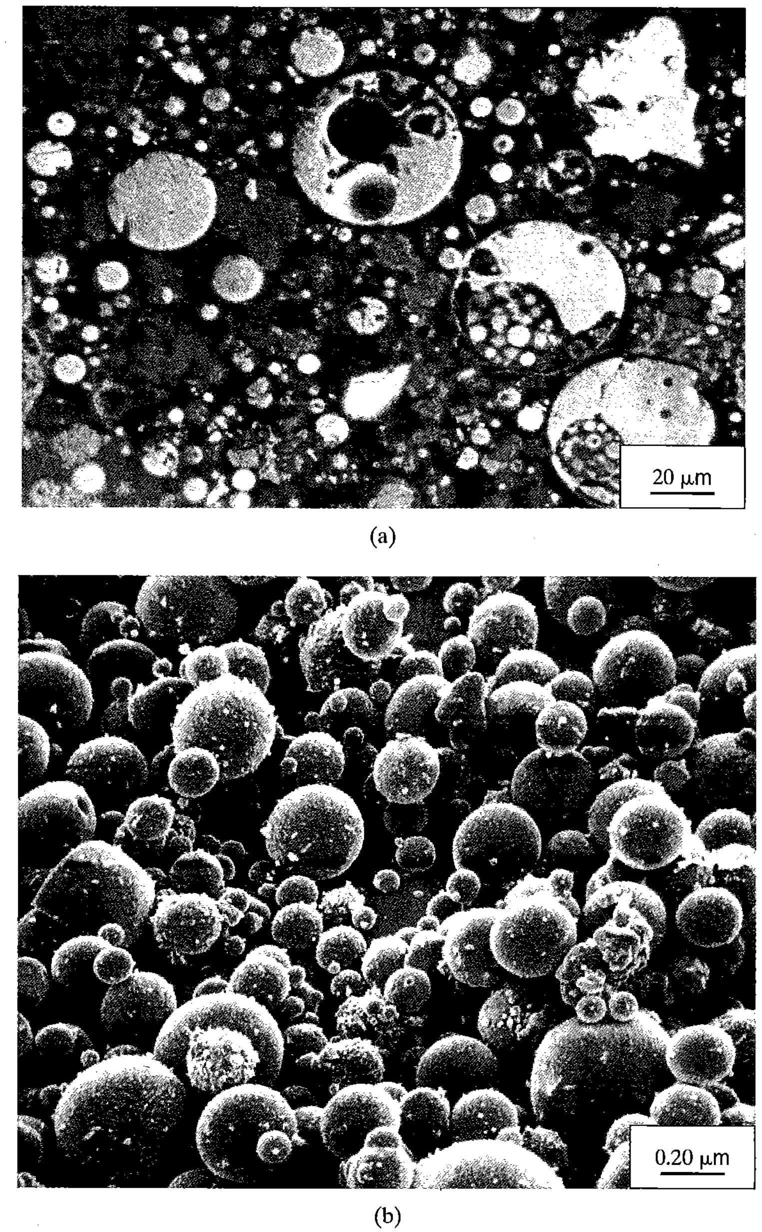


FIGURE 5.3
Particle morphologies of (a) fly ash and (b) silica fume.

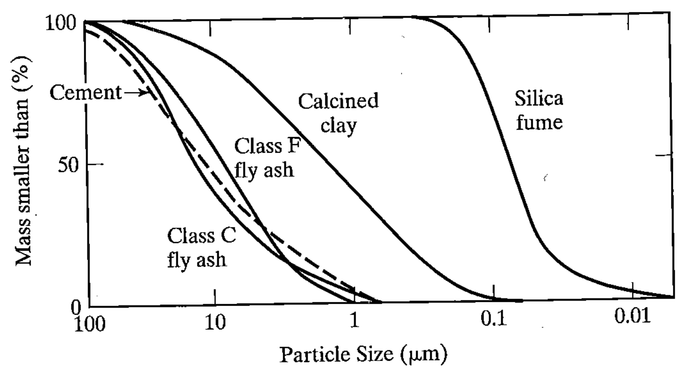


FIGURE 5.4
Typical particle size distribution for various pozzolanic admixtures.

5.2 BLAST FURNACE SLAG

Composition

Slags are residues from metallurgical processes, either from production of metals from ore or refinement of impure metals. They are derived from lime-based inorganic fluxes used to extract impurities from metals, which solidify on cooling. The slags used in concrete come from the blast furnace production of iron from ore. Steel-making slags (from the conversion of iron to steel) or those used to produce nickel or copper are rich in iron and do not have a composition suitable for use in concrete. If beneficiated, however, they can be used. Blast furnace slags are rich in lime, silica, and alumina. Typical mass percentages are CaO = 35–45%; SiO_2 = 32–38%; Al_2O_3 = 8–16%; MgO = 5–15%; Fe_2O_3 < 2%, and sulfur 1–2%. (Residual iron is usually present as ferrous sulfide, rather than as ferric oxide.) ASTM C 989 limits the amount of sulfur as sulfide to 2.5%, and as sulfate to 4.0%.

When cooled slowly in air, the molten slag crystallizes to form inert calcium magnesium silicates, such as melilite, merwinite, and monticellite, and smaller amounts of calcium silicates, such as dicalcium silicate, rankinite (C_3S_2), and pseudowollastonite (CS). Thus, slag must be rapidly cooled (quenched) to form a hydraulically active calcium aluminosilicate glass. Quenching is done using water in one of two processes: granulation or pelletization. In the granulation process, the molten slag is broken up by water jets and then immersed in water. The 4 mm-sized granules are then ground to cement fineness. A newer process, which consumes less water, called pelletizing is shown in Figure 5.5. The smaller granules (<4 mm) are ground for use as a mineral admixture. The larger pellets, which are porous and partially crystalline, can be used as a lightweight aggregate. Therefore, to be hydraulic and hence a useful mineral admixture, blast furnace slag needs to quenched and ground; the term "slag" will be used to imply this state, in the rest of the text.

Reactivities can vary widely from slag to slag. Although it is known that the glass composition is an important factor, there is not a reliable quantitative relationship between slag composition and reactivity. ASTM C 989 assesses reactivity based on determining the strength of mortars made with a 50%–50% slag–cement blend relative to the strength of a pure cement mortar (Table 5.5).

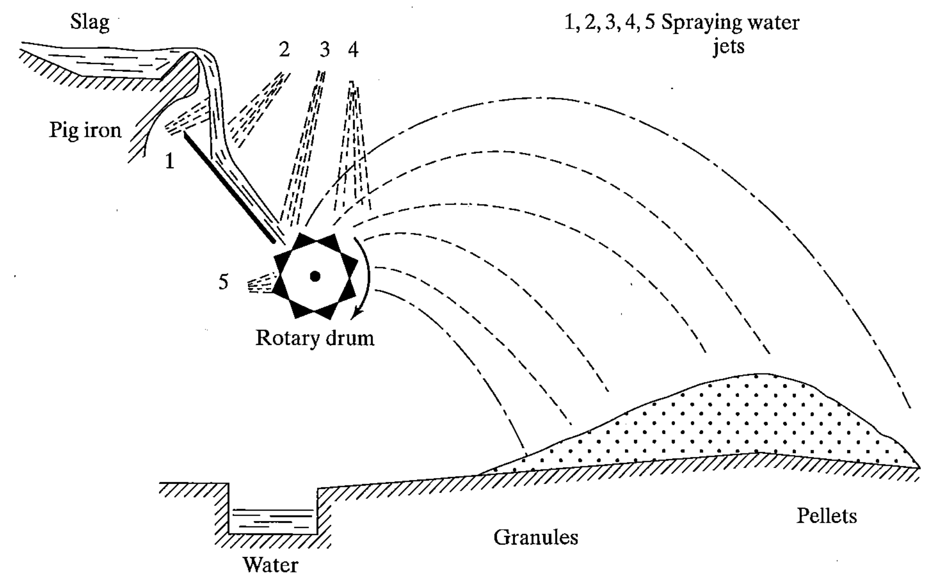


FIGURE 5.5
Schematic view of the pelletizing process for quenching slag. Water jets cool the molten slag before and as the stream is fragmented by the rotating drum. [Adapted from M. Regourd, *Journal of Materials Education*, Vol. 5, No. 5, pp. 687–714(1983).]

TABLE 5.5 Slag Activity Index (ASTM C 989)

	Relative Strength+			
	7 days		28 days	
Designation	*Average**	*Indiv.#*	*Average*	*Indiv.*
Grade 80	—	—	75	70
Grade 100	75	>70	95	>90
Grade 120	95	>90	115	>110

+Percentage of strength of a reference mortar made with pure cement.
*Average of five consecutive samples.
#Any individual sample.

Slag Hydration

The quenched glass reacts slowly with water, so that it may take several months for a pure slag concrete made with slag as the only binder to reach the equivalent 28-day strength of a concrete made with portland cement. This is thought to be due to the presence of impervious coatings of amorphous silica and alumina that form around slag particles early in the hydration process. Slag needs to be activated by alkaline compounds, which can be either soluble sodium salts, such as NaOH, Na_2CO_3, or $NaSiO_3$, or calcium hydroxide. (Calcium sulfates can also activate slags, but are not in common

use). Although there is a lot of interest in using pure slags activated by sodium salts, such systems are only used in specialty applications. Slags are most commonly activated by portland cement, where $Ca(OH)_2$ formed during hydration is the principal activator, with alkali hydroxides playing a minor role. Although only 10–20% of cement is needed for activation, slag–cement blends usually contain much more cement. (see Section 5.3).

The products of slag hydration form a mixture of C–S–H and AFm phases. The composition of the C–S–H is different from that of pure cement having a lower C/S ratio. In slag–cement blends, the slag also shows pozzolanic behavior, since a mixture of low-lime C–S–H and $Ca(OH)_2$ will not be stable, and the C/S ratio of the C–S–H will slowly increase towards a value of 1.7. The rate of hydration of activated slag is similar to that of C_2S, as is the heat of hydration. The activation energy during the early stages of slag hydration is 55 kJ/g, which is higher than that for cement (40 kJ/g).

5.3 BLENDED CEMENTS

Although most pozzolans and slags are used as mineral admixtures, a few cement companies offer portland–pozzolan or portland–slag blended cements. (In Canada, a portland–silica fume blended cement is available.) These must conform either to ASTM specification C 595 or the performance-based specification ASTM C 1157. Table 5.6 summarizes the types of blended cements in C 595. There is little difference in performance in the way the admixture is added, only in the amount added. The use of a blended cement offers convenience and quality control and eliminates the need to handle an extra ingredient at the ready-mixed concrete plant. The disadvantage is that the pozzolan or slag is added at a fixed level, which may not be optimum for a particular application.

While C 595 provides for compositional limits, it is still basically a performance specification, like C 150, using the same tests and the same (or similar) limits on performance. C 1157 is a purely performance-based specification with no compositional limits imposed. In this specification, six types are recognized to meet certain requirements

TABLE 5.6 Blended cements in ASTM C 595

*Type**	*Description*	*Blend levels (mass %)*	*Equivalent C 150 Type*
I (PM)	Pozzolan-modified	<15	II
IP	Portland–pozzolan	15–40	IV or V
I(SM)	Slag-modified	<25	II
IS	Portland–slag	25–70	IV or V
S	Slag	>70	IV or V

*Each type can have additional designations: MS = moderate sulfate resistance, MH = moderate heat of hydration, A = air-entrained.

TABLE 5.7 Types of Hydraulic Cements Designated in ASTM C 1157

Type	*Description*	*Equivalent ASTM C 150*
GU	General Use	I
HE	High Early Strength	III
MS	Moderate Sulfate Resistance	II
HS	High Sulfate Resistance	V
MH	Moderate Heat of Hydration	II
LH	Low Heat of Hydration	IV

TABLE 5.8 Standard Physical Requirements in ASTM C 1157

Cement Type	*GU*	*HE*	*MS*	*HS*	*MH*	*LH*
Fineness[a]						
Autoclave length change, max., %	0.80	0.80	0.80	0.80	0.80	0.80
Time of setting, Vicat test						
Initial, not less than, min.	45	45	45	45	45	45
Final, not more than, max.	420	420	420	420	420	420
Strength range[b], not less than, MPa						
1 day	–	10	–	–	–	–
3 days	10	17	10	5	5	–
7 days	17	–	17	10	10	–
28 days	–	–	–	17	–	17
Heat of hydration, max., kJ/kg						
7 days	–	–	–	–	290	250
28 days	–	–	–	–	–	290
Mortar bar expansion						
14 days, max., %	0.02	0.02	0.02	0.02	0.02	0.02
Sulfate expansion[c], max., %						
6 months	–	–	0.10	0.05	–	–
1 year	–	–	–	0.10	–	–
Option R - low reactivity with alkali-reactive aggregates[d]						
Expansion, max., %						
14 days	0.02	0.02	0.02	0.02	0.02	0.02
56 days	0.06	0.06	0.06	0.06	0.06	0.06
Optional physical requirements						
Early stiffening, final penetration, min.,%	50	50	50	50	50	50
Compressive strength, min., MPa	28	–	28	–	22	–

[a] Both amount retained when wet-sieved on the 45-μm (No. 325) sieve and specific surface area by air permeability, in m^2/g, shall be reported.
[b] Compressive strength. A higher strength range can be specified.
[c] Testing of HS cement at one year shall not be required when the cement meets the six-month limit. A cement shall not be rejected unless it fails both the six-month and one-year limits.
[d] Specify only if the cement will be used with an alkali-reactive aggregate.

(Table 5.7). Neither the category of mineral admixture nor the type of portland scement used in the blend is specified. The specification requirements are given in Table 5.8. ASTM C 1157 represents the type of performance specification that will become increasingly common and may ultimately replace C 150.

5.4 PROPORTIONING WITH MINERAL ADMIXTURES

Mineral admixtures can be used in two ways: as an addition to concrete or as a replacement for some of the portland cement. The addition approach is usually used to improve workability, when minimum cement contents are specified for durability. Replacement is employed to reduce the heat of hydration or to improve durability. When used as an addition, the cement content is kept the same and the pozzolan effectively replaces sand in the concrete mix. When used as a replacement, it substitutes for cement, as is the case when blended cements are used. The sand content may still be affected, however, because replacement is usually made on a mass basis, not by volume. Since the admixtures invariably have lower specific gravities than cement, the replacement admixture will occupy more volume than the cement that it replaces. The *w/c* ratio is calculated on the basis of all materials that contribute to the binding reactions, thus becoming the *w/cm* ratio, where *cm* stands for cementitious material. Mineral and chemical admixtures are frequently used together.

High-Volume Fly Ash Concrete

Slag can be used in large quantities as a cement replacement because of its inherent cementitious properties. A pozzolan, however, is limited by the amount of $Ca(OH)_2$ in the paste with which it can react. There is, theoretically, only enough $Ca(OH)_2$ to react with a Class F fly ash replacing about 25 wt.% of cement. However, it has been found that durable structural concrete can be produced using >50 wt. % of fly ash. The favorable effects of fly ash on workability combined with a water-reducing admixture allow *w/cm* ratios <0.35, which offset the presence of larger quantities of unreacted ash. Residual fly ash acts as a microfiller that is integrally bonded within the hydrated matrix. Although compressive strengths remain low during the first 28 days, long-term strengths are satisfactory (Figure 5.6). This makes these formulations ideal for mass concrete applications where high early strengths are not needed. The low heat of hydration is also advantageous for such concretes.

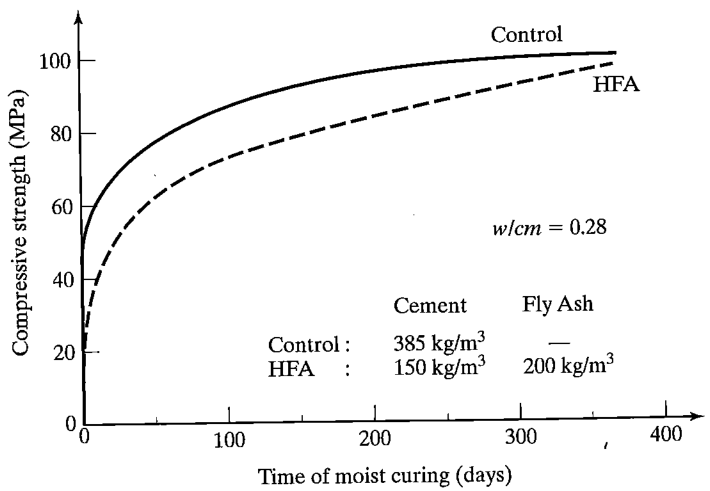


FIGURE 5.6

Comparison of strength developments of plain concrete and concrete with a high fly ash content (HFA).

5.5 EFFECTS ON PROPERTIES OF FRESH AND HARDENING CONCRETE

Heat of Hydration

Mineral admixtures reduce the overall heat of hydration and particularly the rate of heat liberation, which controls the temperature rise in hardening concrete. The use of a mineral admixture is equivalent to raising the C_2S content of the cement and effectively turns a Type I cement into a Type II or Type IV cement. The use of pozzolans in mass concrete has eliminated the production of Type IV cements in the U.S. except for oil-well applications. They are still produced in Canada, however. Although the heat of hydration for the pozzolanic reaction is generally considered to be low, it may actually be higher when normalized on a weight basis. However, since the reaction is slow, heat is released slowly and the temperature rise in mass concrete is invariably reduced (see Figure 5.7).

Workability

Mineral admixtures can be added to increase the cementitious material content, to improve the workability of harsh mixes (e.g., concretes made with coarse or angular sands, or fiber-reinforced concretes), or to achieve high strength without causing high internal temperatures. Because of their spherical morphology, fly ash and silica fume

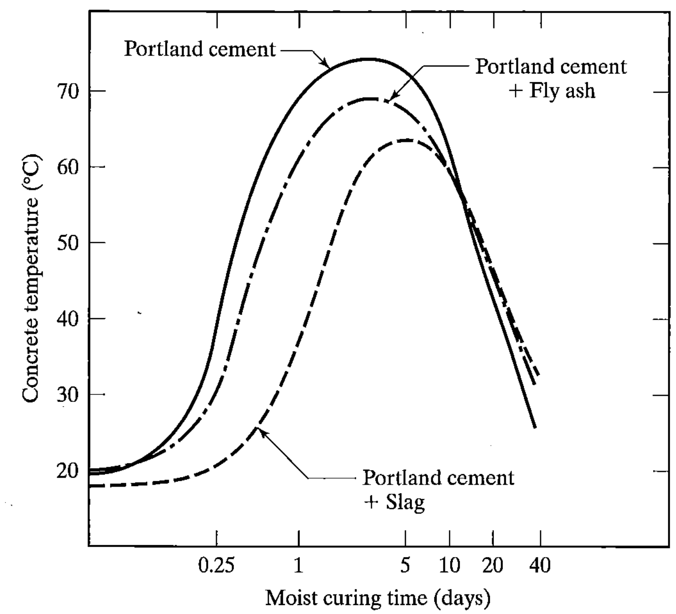


FIGURE 5.7

Temperature rise in mass concrete made with fly ash or slag. Type I cement, $w/c = 0.5$. [Adapted from P. B. Banforth, *Proceedings of the Institution of Civil Engineers*, Part 2, pp. 777–800 (Sept. 1980)].

are particularly beneficial to workability and are considered essential in the production of high-strength concrete. The spherical particles act as small ball bearings to reduce interparticle friction. Thus, the addition of fly ash allows the *w/cm* ratio to be reduced while still maintaining slump. When used in conjunction with a water-reducing agent, *w/c* ratios below 0.35 are possible with fly ash additions of up to 25% by weight of cement.

Silica fume is even more beneficial in this regard, provided a water-reducing admixture is used with it, since chemical dispersion is needed to eliminate the high intrinsic water demand of the sub-micron particles. The silica fume particles are able to pack between the cement grains and act as a very effective lubricant. The use of silica fume is essential to the production of concretes stronger than 100 MPa (15,000 lb/in.^{2}). The fineness of silica fume also makes it very effective in eliminating bleeding and segregation, but makes the concrete very susceptible to plastic shrinkage cracking. Concretes containing silica fume are "sticky" and therefore more difficult to finish.

Air Entrainment

Concretes containing mineral admixtures must be air entrained for frost resistance (see Chapters 8 and 18), unless the *w/cm* ratio is below 0.24 (Chapter 19). The addition of a mineral admixture will often change the amount of air-entraining agent required to achieve a specified air content. Residual carbon strongly inhibits air entrainment when fly ash or rice husk ash is used, but even carbon-free pozzolans can affect air entrainment. Thus, appropriate adjustments must be made when proportioning such concretes.

Microstructure

Mineral admixtures modify the microstructure of the hydrated matrix quite significantly. The increased fraction of C–S–H leads to a more homogenous microstructure, particularly when highly reactive, finely divided pozzolans are used. Residual $Ca(OH)_2$ crystals are usually smaller and can have altered morphology. The principal effect on the microstructure is the change in pore structure. Admixture reactions will reduce the overall porosity (the specific gravity of C–S–H is lower than calcium hydroxide) and the pore size. However, reduction of porosity requires extensive pozzolanic reaction produced by prolonged moist curing. Reductions in the *w/cm* ratio will produce larger reductions in porosity. The particle packing characteristics of silica fume are particularly effective in reducing the pore size, and for this reason, silica fume is used extensively in bridge deck overlays to reduce the rate of chloride penetration to the reinforcing steel. The effects of mineral admixtures on pore size distributions are summarized in Figure 5.8.

Strength Development

Concretes containing mineral admixtures will develop very good strengths over time, exceeding those of similar concretes without admixtures. The extent to which replacement of cement by a mineral admixture will lower early strength depends on its reactivity

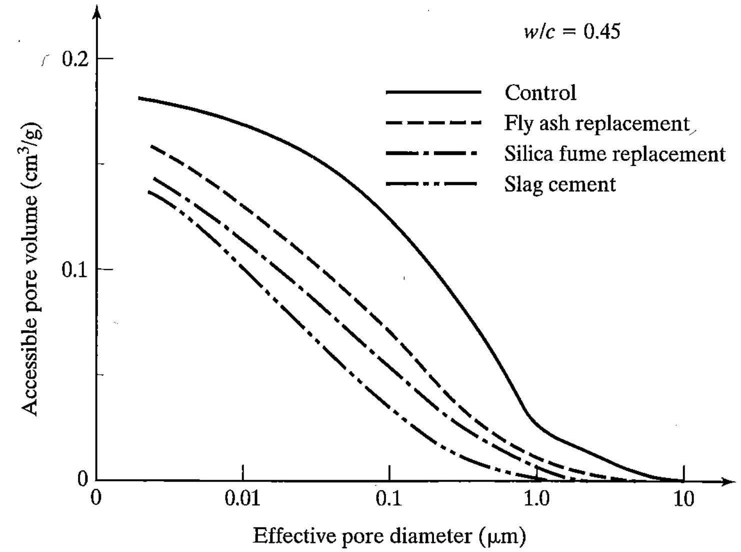


FIGURE 5.8

Comparison of pore size distribution of cement pastes made with mineral admixtures (mercury intrusion porosimetry).

(see Figure 5.9). Reductions in early strength can be offset by lowering the *w/cm* ratio, with the concomitant use of a water-reducing admixture. At early ages, some mineral admixtures essentially behave as an inert addition so that the cement reacts with an effectively high *w/c* ratio. However, highly reactive pozzolans, such as silica fume, calcined clay or rice husk ash, will actually decrease setting times slightly and contribute to early strength. These pozzolans influence cement hydration immediately and start to contribute to strength within the first few days.

Drying Shrinkage and Creep

As a first approximation, it can be stated that the addition of a mineral admixture does not significantly affect the drying shrinkage or creep of concrete. Thus, for most purposes, assuming the same values as used for plain concrete will be satisfactory. Nevertheless, it should be emphasized that if volume changes are critical, then tests should be made to determine the exact characteristics under the anticipated service conditions. Theoretically, one might expect an increase in volume change, due to the increased proportion of C–S–H and the finer pore structure. However, many factors can affect shrinkage and creep values, including the *w/cm* ratio, the amount of admixture used, and the amount of reaction that occurs before drying or loading.

Durability

Mineral admixtures are used extensively for improving the durability of concrete: better sulfate resistance, control of the alkali–silica reaction, decreased chloride diffusion,

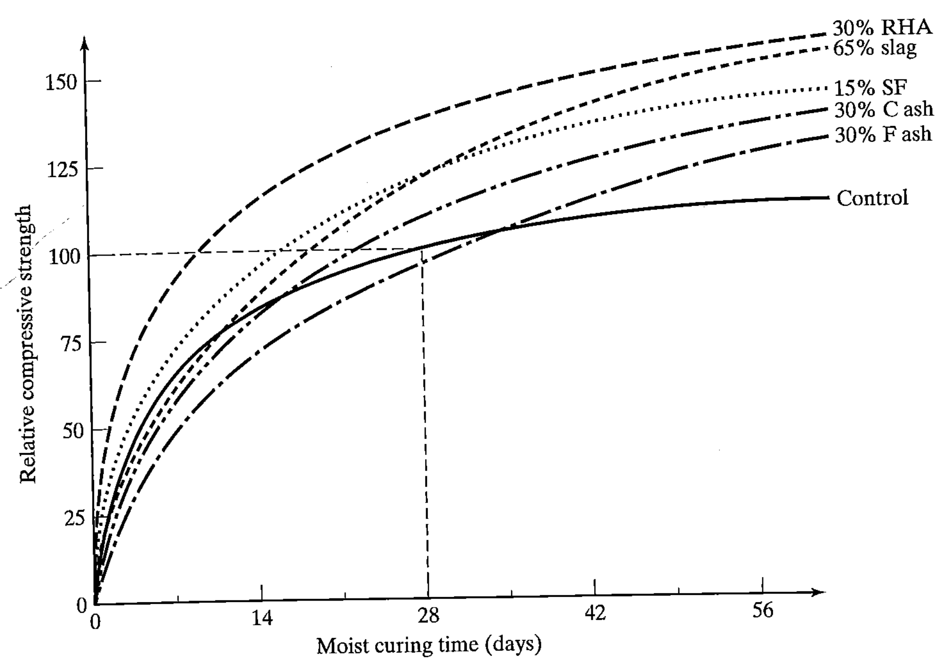


FIGURE 5.9

Comparative strength development of cement–admixture blends.(RHA = rice husk ash; SF = silica fume; C ash = Class C fly ash; F ash = Class F fly ash).

and reduction of leaching and efflorescence. Improvements in durability result from the reduction in calcium hydroxide (which is the most soluble of the hydration products), changes in pore structure, and reductions in the *w/cm* ratio. Natural pozzolans were first used in the U.S. to confer resistance to sulfate attack in structures in California. Pozzolans reduce the amount of ettringite that can form in several ways. Firstly, they reduce the permeability of the concrete and inhibit the ingress of soluble sulfates. Secondly, they reduce the amount of calcium hydroxide, thereby limiting the formation of gypsum from soluble sulfates. Gypsum is a necessary component in the formation of ettringite. Thirdly, their addition as an admixture reduces the effective C_3A content of the cement. The reduction of calcium hydroxide is an important aspect since it can leach out over time, thereby increasing both the porosity and permeability. Sulfate ions can enter the concrete more readily, and calcium hydroxide now becomes a convenient source of calcium ions to concentrate the sulfate ions as gypsum, which eventually react to form ettringite. The reduction of expansion due to sulfate attack by mineral admixtures is shown in Figure 5.10. Using a Type V cement with a mineral admixture can provide even greater resistance.

It is known that not all pozzolans improve sulfate resistance to the same extent, since those with high alumina contents can form C–A–H: compounds [Eq. (5.3)] that will increase the potential ettringite formation. Also, the increased CaO contents in Class C fly ashes will reduce the consumption of calcium hydroxide in the pozzolanic

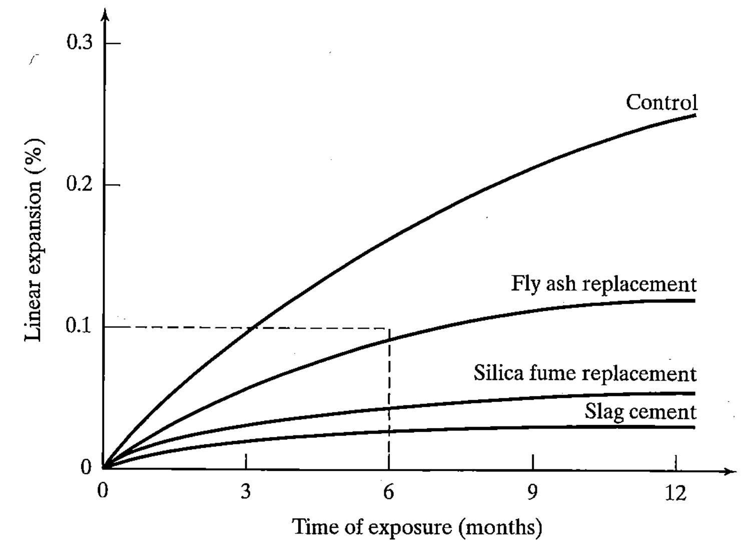


FIGURE 5.10

Effect of mineral admixtures on the linear expansion of concretes exposed to standard ASTM tests for sulfate attack or alkali-aggregate reaction (dashed lines indicate the expansion limits for alkali-aggregate reactions).

reactions. In the 1970s, the Bureau of Reclamation assessed the suitability of fly ashes for improving the sulfate resistance of their concrete structures and proposed an empirical *resistance factor:*

$$R = (CaO\text{-}5)/Fe_2O_3 \quad (5.4)$$

This provides a useful assessment for the efficacy of different fly ashes. Although the Al_2O_3 content is not found explicitly in the equation, its level is generally controlled by the CaO and Fe_2O_3. Fly ashes with low R values give better sulfate resistance, as shown in Figure 5.11.

Mineral admixtures also reduce expansion due to the alkali–silica (alkali–aggregate) reaction (Chapter 7). The relative abilities of different admixtures parallel their influence on sulfate expansion, which is primarily due to the reduction in the alkali content of the cementitious blend and the reduction in pH of the pore solution. Also, the C–S–H formed in the pozzolanic reaction can absorb alkali ions to a greater extent than that formed by hydration of the calcium silicates alone. Admixtures that have high alkali contents may be less effective.

The reduction in pore size results in a general decrease in transport properties, water permeability, and ionic diffusion, which are often the rate-limiting properties in chemical attack. A decrease in chloride diffusion is the major reason for the reduction in chloride penetration when mineral admixtures are used. Representative data are shown in Figure 5.12.

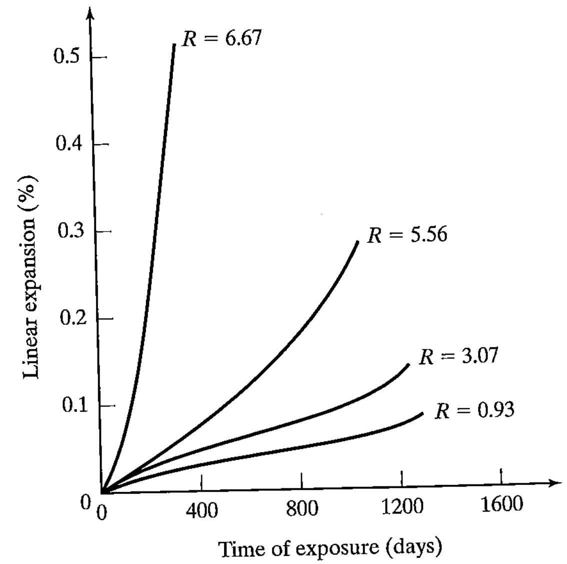


FIGURE 5.11

Sulfate expansion of concretes made with Class C fly ash.[Adapted from E. R. Dunstan, *Cement Concrete and Aggregates*, Vol. 2, pp. 20–30, (1980).]

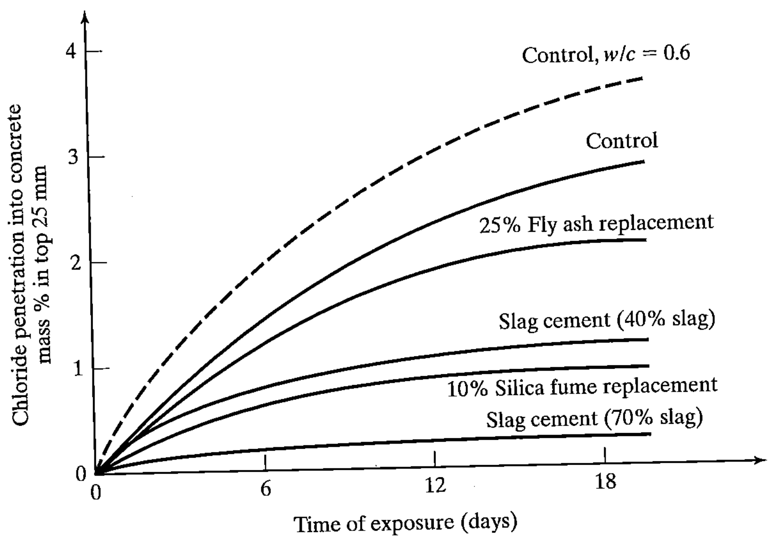


FIGURE 5.12

Comparison of chloride penetration into concretes (w/c = 0.5) made with mineral admixtures.

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PROBLEMS

5.1. What are the important differences between silica fume and Class F fly ash?
5.2. Why do some class C fly ashes have cementitious properties? Is this behavior advantageous?
5.3. Describe the difference between the pozzolanic reaction and the slag reaction.
5.4. What is the purpose of the process of pelletization or granulation of blast furnace slag?
5.5. Describe the benefits of using fly ash to improve the properties of fresh concrete.
5.6. Why is the addition of blast furnace slag rather than fly ash a better strategy to improve sulfate resistance of concrete?
5.7. What class of blended cements would you specify for the following applications? (a) mass concreting; (b) moderate sulfate resistance; (c) protection against alkali–aggregate reaction; (d) protection of reinforcing steel from corrosion; (e) high-sulfate resistance.
5.8. What is the major difference between ASTM standards C 595 and C 1157?

CHAPTER 6

Water

Water is a key ingredient in the manufacture of concrete. It is also a material in its own right. Understanding its properties is helpful in gaining an understanding of its effects on concrete and other building materials. Although it is critical to the construction process, water has properties that make it an enemy of highways and structures: It causes freeze–thaw damage and plays a key role in corrosion. It is also a highly effective solvent and may cause damage due to moisture migration as it moves from warm to cold regions in a structure or structural member. In this chapter, we briefly discuss the properties of water as a material and then describe specific requirements for its use in concrete.

6.1 WATER AS A MATERIAL

The Water Molecule

The chemical combination of two atoms of hydrogen with one atom of oxygen results in the formation of a molecule with hybrid orbitals that give the material special properties. As shown in Figure 6.1, the four orbitals, each with two electrons, are oriented in two perpendicular planes. The two orbitals with the hydrogen atoms, the bond hybrids, behave as if they have a net positive charge. The other two, the lone pair hybrids, have a net negative charge. The distance between the nuclei of the oxygen and hydrogen atoms is approximately 1×10^{-10} m. The differences in charge cause the lone pair hybrids on one water molecule to be attracted to bond hybrids on other water molecules, giving rise to tetrahedral structural coordinations (i.e., combinations of one atom with four others) in ice and liquid water. The strong attraction between lone pair and bond hybrid orbitals results in the formation of *hydrogen bonds*, which provide high cohesive forces between molecules. Hydrogen bonds are the principal reason that water has high heats of fusion (334 J/g or 79.7 cal/g) and vaporization (2258 J/g or 539 cal/g), as well as a high specific heat (4.18 J/g/°C or 1 cal/g/°C). Water forms polar bonds with other materials.

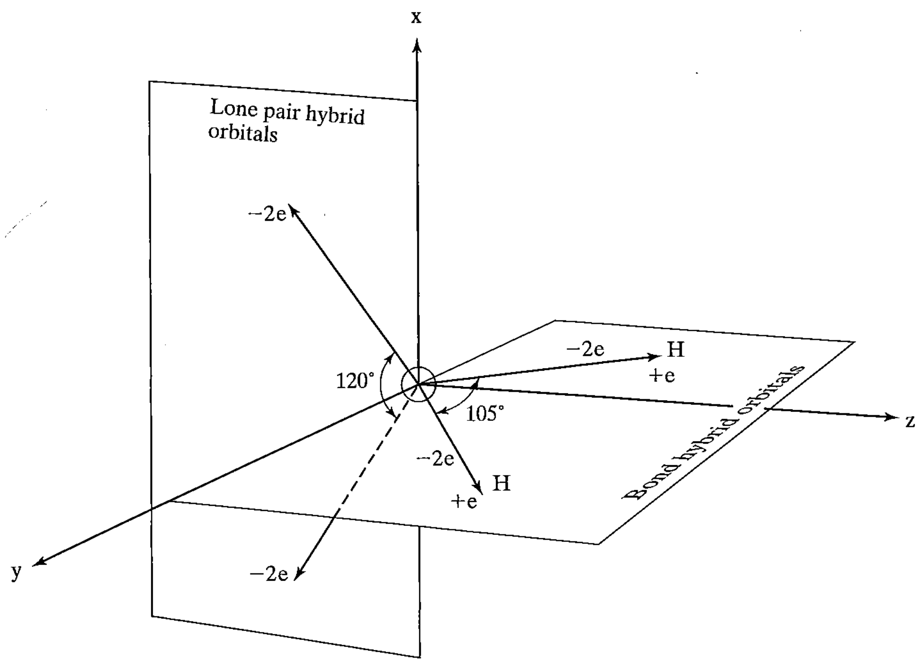


FIGURE 6.1
Model of water molecule, showing orientations of hybrid orbitals.

Ice

The crystal structure of solid water is a function of both temperature and pressure. Different crystal structures are referred to as polymorphs. The first polymorph that forms at atmospheric pressure is known as ice I. The crystal is held together by hydrogen bonds, with each lone pair orbital bonding with the H-O of another molecule, giving O–H-O, and each hydrogen ion bonding with a lone pair, giving O-H–O. Thus, every water molecule has four neighbors. The resulting crystal is illustrated in Figure 6.2, with its well-known hexagonal structure. The crystal contains vacant "shafts" that cause a decrease in density and an increase in volume (9%) compared to the liquid; the center-to-center spacing of the oxygen atoms is 2.76×10^{-10} m. The increase in volume allows ice to float on its own melt, a property shared with only a few other materials (diamond, silicon, germanium).

The freezing temperature of water, 0°C (32°F) at standard pressure, decreases as pressure increases. For water located in another material, such as concrete, the freezing temperature also is a function of the void or pore size, decreasing as the size of the void or pore decreases. As a result, very little of the free water in concrete freezes at 0°C. Even when the freezing temperature is reached, ice will not form unless there is a "seed" present. Without a seed, water supercools, remaining in liquid form. When frozen, water is pure; that is, any solute atoms that are present in the liquid are not combined in the crystal structure. Rather, they are left in a more concentrated solution adjacent to the crystal. For example, sea ice consists of pure water, in the form of ice,

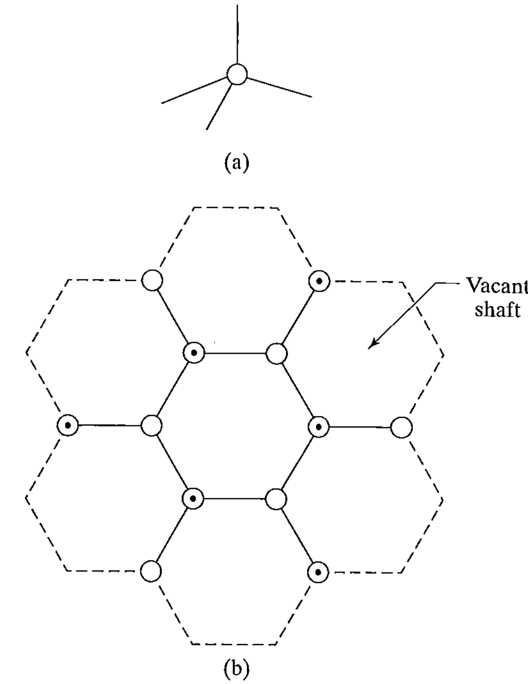


FIGURE 6.2
(a) Representation of water molecule; (b) hexagonal crystalline structure of ice as viewed along the c crystal axis.

with a salt-laden brine that is not part of the crystal structure. The purity of ice can be used to advantage as one method of purifying water.

Liquid Water

In liquid water, each molecule is coordinated with four others, at room temperature and below. The clusters of molecules are constantly breaking up and reforming. As ice melts, increased vibrational motion tends to increase volume, while a more compact structure, due to collapse of the crystal (with its vacant shafts), results in a decrease in volume. From 0 to 4°C, the effect of the more compact structure predominates; water is densest at 4°C. Above 4°C, the effect of the increased vibrational motion takes over, and volume increases. These changes in volume are rather small compared to the reduction in volume as crystalline ice melts. The volume drops 0.013% from 0 to 4°C and increases by 0.31% as the temperature rises from 4 to 100°C. Above 25°C, the tetrahedral coordination breaks down, progressively, until individual molecules escape as a gas at 100°C.

Water is a strong solvent and readily dissolves building materials such as gypsum and lime. The lime (CH) produced in the hydration reaction can, thus, be leached out of concrete.

6.2 WATER QUALITY

Although water is an important ingredient of concrete, little needs to be written about water quality, since it has little to do with the quality of concrete. There is much more bad concrete made through using too much good-quality water than there is using the

right amount of poor-quality water. The time-honored rule of thumb for water quality is, "If you can drink it, you can make concrete with it." A large fraction of concrete is made using municipal water supplies. However, good-quality concrete can be made with water that would not pass normal standards for drinking water.

Impurities in Water

Mixing water can cause problems by introducing impurities that have a detrimental effect on concrete quality. Although satisfactory strength development is of primary concern, impurities contained in the mix water may also affect setting times, drying shrinkage, or durability, or they may cause efflorescence. Water should be avoided if it contains large quantities of suspended solids, excessive amounts of dissolved solids, or appreciable amounts of organic materials. Concentration limits for various impurities are given in Table 6.1.

Suspended Solids Usually up to about 2000 ppm of suspended clay or silt can be tolerated. Higher amounts may increase water demand, increase drying shrinkage, or cause efflorescence. Muddy water should be allowed to clear in settling basins before use. Algae or other suspended organic matter may also be a problem, as they will not settle out readily on standing. Organic materials may dissolve during mixing (since the mix water quickly becomes highly alkaline) and subsequently retard setting and

TABLE 6.1 Tolerable Levels of Impurities in Mixing Water

Impurity	*Maximum Concentration s(ppm)*	*Remarks*
Suspended matter (turbidity)	2000	Silt, clay, organic matter
Algae	500–1000	Entrain air
Carbonates	1000	Decrease setting times
Bicarbonates	400–1000	400 ppm for bicarbonates of Ca, Mg
Sodium sulfate	10,000	May increase early strength, but reduce later strength
Magnesium sulfate	40,000	
Sodium chloride	20,000	Decrease setting times, increase early strength, reduce ultimate strength
Calcium chloride	~50,000	
Magnesium chloride	40,000	
Iron salts	40,000	
Phosphates, arsenates, borates	500	Retard set
Salts of Zn, Cu, Pb, Mn, Sn	500	
Inorganic acids	10,000	pH not less than 3.0
Sodium hydroxide	500	
Sodium sulfide	100	Should test concrete
Sugar	500	Affects setting behavior

strength development by interfering with cement hydration. They may also entrain excessive amounts of air, thereby reducing strength. Conversely, they may interfere with the action of air-entraining agents.

Dissolved Solids Water containing less than 2000 ppm of dissolved solids can in most instances be used safely, although this depends, of course, on the nature of the dissolved material. As little as 100 ppm of sodium sulfide may cause problems. At the other extreme, seawater [which contains about 34,000 ppm (3.4%) of dissolved salts] can be used to make satisfactory concrete if certain precautions (described in the next section) are taken. Indeed, soluble salts may be added deliberately as admixtures—the most common example being $CaCl_2$, which is used as an accelerating agent. Soluble carbonates and bicarbonates can promote rapid setting; large quantities of carbonates and sulfates may cause a reduction in 28-day strength or long-term strength.

Some soluble inorganic salts may retard the setting and hardening of concrete. Salts of zinc, copper, lead, and, to a lesser extent, manganese and tin fall into this category, as well as phosphates, arsenates, and borates. Up to 500 ppm can generally be tolerated in mixing water. Such compounds are likely to be found at these concentrations only in untreated industrial wastewaters, although waters leached from a mining locale may contain significant quantities of metal salts.

Acidic waters can be used in concrete making; the pH of the water may be as low as 3.0, at which level there are more problems surrounding the handling of the water than will occur in the concrete. Organic acids may affect the setting and hardening of concrete. Alkaline waters, containing sodium or potassium hydroxide, may cause quick setting and low strengths at concentrations above 500 ppm.

Seawater Seawater contains dissolved salts, most of which are sulfates and chlorides of sodium and magnesium (Table 6.2). Thus, more rapid setting and early strength gain can be expected, owing to the accelerating effect of the chloride ion (see Chapter 8), but the 28-day strength will be lower because of the higher amounts of sulfate, which will prolong the crystallization of ettringite. A strength loss of 10 to 20% will be typical and can be compensated for by using a lower *w/c* ratio. The presence of chloride ions increases the risk of corrosion of reinforcing steel, and hence seawater should *never* be used for making pre-stressed concrete. The use of seawater is allowed for ordinary reinforced concretes. If an adequate protective cover of good, dense concrete is provided, problems with corrosion should be minimal. However, it may be more difficult to protect reinforcement reliably in concrete made with seawater than is often assumed. Thus, it is best to avoid using seawater for reinforced and architectural concrete unless

TABLE 6.2 Typical Composition of Seawater (ppm)

Sodium chloride (NaCl)	27,000
Magnesium chloride ($MgCl_2$)	3,200
Magnesium sulfate ($MgSO_4$)	2,200
Calcium sulfate ($CaSO_4$)	1,100
Calcium chloride ($CaCl_2$)	500
Total dissolved salts	34,000

it is unavoidable. The use of seawater causes efflorescence and may cause problems with decorative finishes.

Dissolved Organic Material Colored natural waters generally indicate the presence of dissolved organic material (mostly tannic and humic acids), which may retard the hydration of cements. Many organic compounds that occur in industrial wastes may also severely affect the hydration of cement or entrain excessive amounts of air. Wastes from the pulp and paper industries, tanning industries, and food-processing industries have been used as a source of chemicals for the formulation of set-retarding or air-entraining admixtures. Thus, untreated industrial wastewaters should be viewed with caution, but if they have passed through a sewage-treatment process, organic matter will be reduced to safe levels.

Testing of Water

Potable water need not be tested prior to its use in concrete. ASTM C 94 and the ACI Building Code require tests when the water is not potable. In both standards, water is evaluated based on its effect on the strength of mortar cast in accordance with ASTM C 109. In addition, ASTM C 94 limits the effect on setting time (measured by ASTM C 191) to not more than one hour earlier nor more than one-and-a-half hours later than a control specimen made with potable water. In ASTM C 94, a mortar specimen made with "questionable" water (i.e., water that "contains quantities of substances which discolor it or make it smell or taste unusual or objectionable or cause suspicion") must produce a 7-day strength equal to at least 90% of the strength obtained with a control specimen produced using potable water. The ACI Building Code allows the use of nonpotable water providing that concrete proportions are selected based on mixes using water from the same source and that mortar cubes made with the nonpotable water provide both 7 and 28-day strengths equal to at least 90% of the strengths of similar specimens made with potable water.

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PROBLEMS

6.1. Why does water expand when it freezes?

6.2. Should seawater be used to make reinforced concrete? Discuss the problems that may arise.

6.3. Is it advisable to use industrial wastewaters for concrete mixing?

6.4. Can wastes containing large amounts of suspended inorganic materials be used to make concrete?

CHAPTER 7

Aggregates

Aggregates generally occupy 70 to 80% of the volume of concrete and can therefore be expected to have an important influence on its properties. They are granular materials, derived for the most part from natural rock (crushed stone, or natural gravels) and sands, although synthetic materials such as slags and expanded clay or shale are used to some extent, mostly in lightweight concretes. In addition to their use as an economical filler, aggregates generally provide concrete with better dimensional stability and wear resistance. The influences that the aggregates can have on the mechanical and physical properties of concrete (see Table 7.1) are discussed in succeeding chapters. Here we will discuss the influence of aggregate properties on mix design and special durability problems associated with aggregate. Although aggregate strength can play an important role in high-strength concretes, for most applications the strength of concrete and mix design are essentially independent of the composition of aggregates. Durability, however, may be affected. No particular rock or mineralogical type, in itself, is required for aggregates, but it has been found that certain constituents may cause problems in the field, as will be discussed later. In other instances, a certain kind of rock may be required to attain certain concrete properties (e.g., high density or low coefficient of thermal expansion). But in the absence of special requirements, most kinds of rocks can produce acceptable aggregates that conform to ASTM C 33. Thus, aggregates are not generally classified by mineralogy; the simplest and most useful classification is on the basis of specific gravity. (The geology of rocks will not be considered in this text. ASTM C 294 describes principal rock types and constituents of common aggregates.) Aggregates are classed as heavyweight, normal-weight, and lightweight. Most of this chapter is devoted to normal-weight aggregates, which make up about 90% of the concrete produced in the United States and Canada, since the same considerations apply generally to the other weight classes.

Aggregates should be hard and strong, free of undesirable impurities, and chemically stable. Soft, porous rock can limit strength and wear resistance; it may also break down during mixing and adversely affect workability by increasing the amount of fines. Rocks that tend to fracture easily along specific planes can also limit strength and wear resistance. Therefore, it is best to avoid aggregates that contain a significant proportion of weak or friable materials, or to remove these. Aggregates should also be free of

TABLE 7.1 Properties of Concrete Influenced by Aggregate Properties

Concrete Property	*Relevant Aggregate Property*
Durability	
Resistance to freezing and thawing	Soundness, porosity, pore structure, permeability, degree of saturation, tensile strength, texture and structure, clay minerals
Resistance to wetting and drying	Pore structure, modulus of elasticity
Resistance to heating and cooling	Coefficient of thermal expansion
Abrasion resistance	Hardness
Alkali–aggregate reaction	Presence of particular siliceous constituents
Strength	Strength, surface texture, cleanness, particle shape, maximum size
Shrinkage and creep	Modulus of elasticity, particle shape, grading, cleanness, maximum size, presence of clay minerals
Coefficient of thermal expansion	Coefficient of thermal expansion, modulus of elasticity
Thermal conductivity	Thermal conductivity
Specific heat	Specific heat
Unit weight	Specific gravity, particle shape, grading, maximum size
Modulus of elasticity	Modulus of elasticity, Poisson's ratio
Skid resistance	Tendency to polish
Economy	Particle shape, grading, maximum size, amount of processing required, availability

impurities: silt, clay, dirt, or organic matter. If these materials coat the surfaces of the aggregate, they will isolate the aggregate particles from the surrounding concrete, causing a reduction in strength. Silt and clay and other fine materials will also increase the water requirements of the concrete, and organic matter may interfere with cement hydration.

Standard tests used in the United States and Canada to evaluate aggregates are listed in the Appendix.

7.1 PROPERTIES REQUIRED FOR MIX DESIGN

To proportion suitable concrete mixes, certain properties of the aggregate must be known: (1) shape and texture, (2) size gradation, (3) moisture content, (4) specific gravity, and (5) bulk unit weight. These properties influence the paste requirements for workable fresh concrete.

Shape and Texture

Effect on Workability Aggregate shape and texture affect the workability of fresh concrete through their influence on cement paste requirements. Sufficient paste is required to coat the aggregates and to provide lubrication to decrease interactions between

aggregate particles during mixing. For good workability, the ideal aggregate particle is one that is close to spherical in shape (well rounded and compact) with a relatively smooth surface (see Figure 7.1). Most natural sands and gravels come close to this ideal. Crushed stone is much more angular (with sharp edges and corners) and may have a rough surface texture. Such particles interfere more severely with the movement of adjacent particles, even when nearly uniform (chunky) in shape. They also have a higher surface-to-volume ratio and therefore require more paste to fully coat the surface of each particle. Highly irregular particles with reentrant faces and sharp points will lead to greater interparticle interactions during mixing and handling. Aggregates that are flat or elongated should be avoided because they increase interparticle interaction and the surface-to-volume ratio, thereby increasing the paste requirements. Also, concretes containing aggregates of this shape are more prone to segregation during handling. The presence of flat or elongated particles in crushed rock may be indicative of rock with weak fracture planes. The surface texture of the aggregate is also important for workability, since a rough surface requires more lubrication for movement. Crushed stone has a rougher surface than natural sand and gravel because the surface has not been worn smooth by the effects of water and weather. There are no specified tests or detailed definitions prescribed by ASTM for surface texture and particle shape, although ASTM C 1252 does promote an indirect measure of both by measuring their effect on the void content of fine aggregate. However, a British standard has a classification system (given in Table 7.2) with useful reference terms.

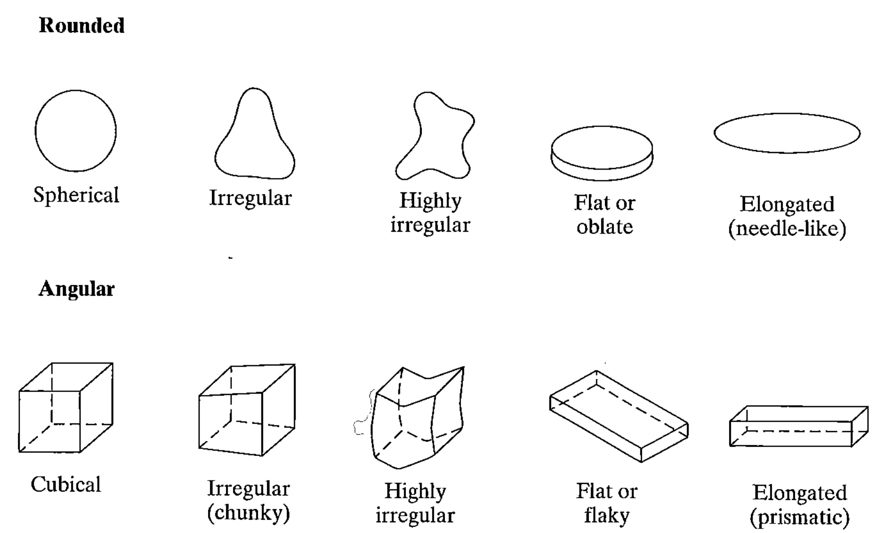


FIGURE 7.1
Classification of aggregate shapes.

TABLE 7.2 Particle Shape Classification and Surface Texture of Aggregates for Concrete[a]

Particle Shape Classification

Classification	*Description*	*Examples*
Rounded	Fully waterworn or completely shaped by attrition	River or seashore gravel; desert, seashore, and windblown sand
Irregular	Naturally irregular, or partly shaped by attrition and having rounded edges	Other gravels; sand or dug flint
Angular	Possessing well-defined edges formed at the intersection of roughly planar faces	Crushed rocks of all types; talus; crushed slag
Flaky	Material of which the thickness is small relative to the other two dimensions	Laminated rock
Elongated	Material, usually angular, in which the length is considerably larger than the other two dimensions	—
Flaky and elongated	Material having the length considerably larger than the width, and the width considerably larger than the thickness	—

Surface Texture of Aggregates

Group	*Surface Texture*	*Characteristics*	*Examples*
1	Glassy	Conchoidal fracture	Black flint, obsidian, vitreous slag
2	Smooth	Waterworn, or smooth due to fracture of laminated or fine-grained rock	Gravels, chert, slate, marble, some rhyolites
3	Granular	Fracture showing more-or-less uniform rounded grains	Sandstone, oolite
4	Rough	Rough fracture of fine or medium-grained rock containing no easily visible crystalline constituents	Basalt, felxite, porphyry, limestone
5	Crystalline	Containing easily visible crystalline constituents	Granite, gabbro, gneiss
6	Honeycombed	With visible pores and cavities	Brick, pumice, foamed slag, clinker, expanded clay

[a]From British Standard (BS) 812: Part 1, 1975.

Effect on Mechanical Properties The shape and texture of the fine aggregate affect only workability, but the characteristics of the coarse aggregate may also affect the mechanical properties of the concrete by affecting the mechanical bond. Shape can favorably influence strength (especially tensile strength) by increasing the amount of

surface area available for bonding with the paste for a given aggregate content. However, extremes in aggregate shape may lead to high internal stress concentrations and lower strength. Rough, textured surfaces will improve the mechanical component of the bond. It is also thought that the mineralogical character of the aggregate can play a role in determining the strength of the aggregate–paste bond, since a chemical interaction may occur at the interface between certain kinds of rock and cement paste.

Size Gradation

The particle size distribution or *grading* of an aggregate supply is an important characteristic because it determines the paste requirements for a workable concrete. Since cement is the most expensive component, it is desirable to minimize the cost of concrete by using the smallest amount of paste consistent with the production of a concrete that can be handled, compacted, and finished and provide the necessary strength and durability. The significance of aggregate gradation is best appreciated by considering concrete as a slightly compacted assembly of aggregate particles bonded together with cement paste, with the voids between particles completely filled with paste. Thus, the amount of paste depends on the amount of void space that must be filled and the total surface area of the aggregate that must be coated with paste. The volume of the voids between roughly spherical aggregate particles is greatest when the particles are of uniform size (Figure 7.2a). When a range of sizes is used, the smaller particles can pack between the larger (Figure 7.2b), thereby decreasing the void space and lowering paste requirements. Using a larger maximum aggregate size (Figure 7.2c) can also reduce the void space. For a given maximum aggregate size, a theoretical grading curve for minimum void space can be worked out using simple particle geometries. Such size distributions do not give a workable concrete; a compromise has to be worked out between workability and economy. This is discussed in more detail in Chapter 10.

Size Analysis The grading of an aggregate supply is determined by a *sieve analysis.* A representative sample of the aggregate is passed through a stack of sieves arranged in order of decreasing size of the openings of the sieve. It is convenient to divide aggregate into coarse and fine fractions. The *coarse aggregate* fraction is that retained on the 4.75-mm sieve (No. 4), while the *fine aggregate* fraction is that passing the same sieve. The common sieve sizes are given in Table 7.3; the standard sieve designation is based on the nominal opening in millimeters and micrometers. The alternate designation is based on the size of the openings in inches for coarse sieves and the number of openings per inch for fine sieves. It can be seen also that each sieve used to grade the fine aggregate has openings that are half the dimension of the next larger one. In the coarse sieves, the ones that fit in this size sequence are the 9.5 mm, 19 mm, 37.5 mm, and 75 mm (3/8 in., 3/4 in., 1 1/2 in., and 3 in.). The sieves in this size sequence are in the *standard series;* the others are called *half sizes.*

Maximum Aggregate Size The maximum size of the coarse aggregate influences the paste requirements of the concrete, and the optimum grading of the coarse aggregate depends on the maximum aggregate size. As defined by ASTM C 125, the maximum

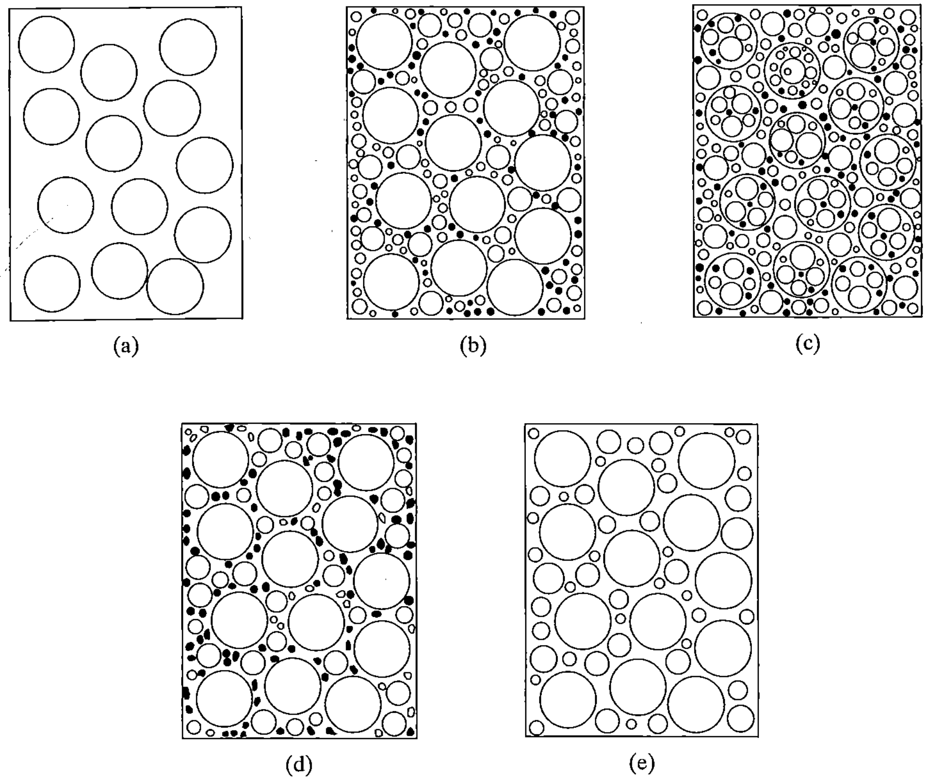


FIGURE 7.2

Schematic representations of aggregate gradations in an assembly of aggregate particles: (a) uniform size; (b) continuous grading; (c) replacement of small sizes by large sizes; (d) gap-graded aggregate; (e) no-fines grading.

size of coarse aggregate is the smallest sieve opening through which the entire sample passes. In practice, it is considered that if only a small amount of aggregate is retained on a sieve, it will not significantly affect the properties of the concrete. Thus, it is usual to use a *nominal* maximum size, which is the smallest sieve opening through which the entire sample is permitted to pass, but need not do so. A percentage (usually 5–10%) of the sample weight may be retained on this sieve. ASTM grading requirements are based on nominal maximum size.

The choice of nominal maximum aggregate size is determined by job conditions. If the aggregate size is too large, the concrete at any given cross section of a member may not be representative of the entire material because of the location of an overly large aggregate particle. To guard against this possibility, the maximum size should not be greater than one-fifth of the smallest dimension between the sides of forms. For slabs, this figure may be relaxed to one-third of the thickness. Large aggregate particles may also get obstructed at narrow openings between reinforcing bars, or between bars and formwork, and cause undesirable segregation during placement. To avoid this problem, the nominal maximum size should not exceed three-fourths of the minimum

TABLE 7.3 ASTM Sieve Sizes Commonly Used for Sieve Analysis of Concrete Aggregates

	ASTM Sieve Designation	*Alternative*	*Nominal Size of Sieve Opening, in.*
Coarse aggregate	75 mm	3 in.	3
	63 mm[a]	$2\frac{1}{2}$ in.	2.5
	50 mm[a]	2 in.	2
	37.5 mm	$1\frac{1}{2}$ in.	1.5
	25 mm[a]	1 in.	1
	19 mm	$\frac{3}{4}$ in.	0.75
	12.5 mm[a]	$\frac{1}{2}$ in.	0.50
	9.5 mm	$\frac{3}{8}$ in.	0.375
Fine aggregate	4.75 mm	No. 4 ($\frac{3}{16}$ in.)	0.187
	2.36 mm	No. 8	0.0937
	1.18 mm	No. 16	0.0469
	600 μm[b]	No. 30	0.0234
	300 μm	No. 50	0.0124
	150 μm	No. 100	0.0059

[a]Half sizes.
[b]1000 μm = 1.0 mm.

clear spacing between reinforcing bars or between reinforcing bars and forms. These two requirements effectively limit the maximum size on most jobs to 40 mm (1 1/2 in.) or less. Larger sizes can be used in unreinforced mass concrete where limitations on size do not apply. Most equipment in the ready-mixed concrete industry can handle up to 50-mm (2 in.) maximum size aggregate, but equipment used in mass concrete construction can handle up to 150-mm (6 in.) coarse aggregate where the large volumes of concrete justify the use of larger equipment needed for the large aggregate sizes. However, most equipment for testing concrete is designed to handle only up to 37.5-mm (1 1/2 in.) maximum size aggregate, so the larger sizes must be screened out before testing.

The higher the maximum aggregate size, the lower the paste requirements for the mix (see earlier discussion). For a given workability and cement content, the strength of concrete increases with increasing aggregate size because the *w/c* ratio can be lowered (Figure 7.3). However, with the larger aggregate sizes, increased internal stresses tend to lower the strength. This effect is noticeable only in rich mixes; concretes with low cement contents, which are typical of mass concrete, show continuing increases in strength. If the *w/c* ratio is not lowered, strength will drop with an increase in the maximum aggregate size. As a result, achieving high concrete strengths [above 70 MPa (10,000 psi)] may require the use of a maximum aggregate size as small as 9.5 mm (3/8 in.). This is discussed in more detail in Chapter 19.

Generally, an increase in the maximum aggregate size will improve concrete durability because there will be less paste subject to chemical or physical attack. However, if an aggregate is susceptible to freeze–thaw damage, a reduction in aggregate size will improve durability, as discussed in Chapter 18.

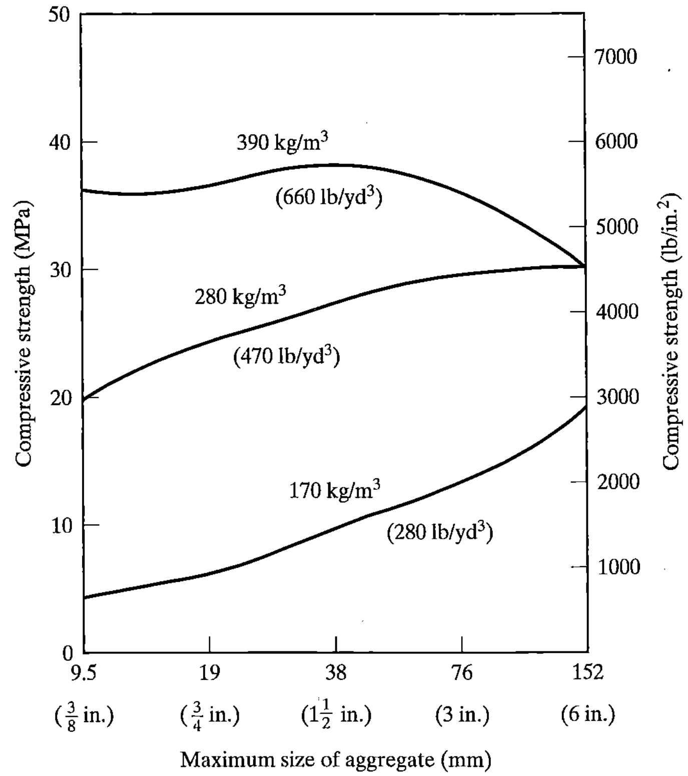


FIGURE 7.3

Influence of aggregate size on 28-day compressive strength of concretes with different cement contents. [From *Symposium on Mass Concrete*, SP-6, American Concrete Institute, Detroit, MI, pp. 219–256 (1963).]

Grading Curves When a sieve analysis has been completed (according to the provisions of ASTM C 136), the weight of aggregate retained on each sieve is expressed as a percentage of the total weight of the sample. This is then used to calculate the cumulative percentage retained on each successive sieve or the cumulative percentage passing each sieve (Table 7.6). These numbers can then be plotted graphically against sieve size, as in Figure 10.1 (Section 10.2), to give a grading curve. It is customary to use cumulative percentage passing on the ordinate. The successive standard sieve sizes are plotted linearly along the abcissa, and this effectively gives a semilog plot, since each sieve size is related by a factor of 2. Therefore, the *half sizes* should be placed between the other sizes as is correct for a logarithmic scale.

ASTM C 33 sets grading limits for fine and coarse aggregate based on practical experience. These limits are summarized in Table 7.4 for fine aggregate and in Table 7.5 for coarse aggregates of various nominal maximum sizes. It can be seen that quite wide grading limits are provided, which allows for adjustment for differences in aggregate shape and texture. If an aggregate does not conform to the ASTM C 33 grading limits, it does not necessarily mean that concrete cannot be made with the aggregate. It does mean that the concrete may require more paste and is more liable to segregate during handling and placing.

It is easiest to maintain uniformity of concrete if the aggregate grading remains constant. On large jobs where considerable quantities of concrete are to be handled, it

may be advantageous to blend several fractions of coarse aggregate to maintain uniform grading. This is a common procedure in ready-mixed concrete plants, since concrete can be produced more economically when corrections for grading variability are eliminated.

Fineness modulus. The use of a single parameter to describe the grading curve can be useful in checking the uniformity of grading. The *fineness modulus* is such a parameter. It is defined as

$$\text{FM} = \frac{\Sigma\,(\text{cumulative percent retained on standard sieves})}{100} \tag{7.1}$$

TABLE 7.4 ASTM Grading Limits for Fine Aggregates[b]

Sieve Size	*% Passing*[a]
9.5 mm ($\frac{3}{8}$ in.)	100
4.75 mm (No. 4)	95–100
2.36 mm (No. 8)	80–100
1.18 mm (No. 16)	50–85
600 μm (No. 30)	25–60
300 μm (No. 50)	10–30
150 μm (No. 100)	2–10

[a] Not more than 45% should be retained between any two consecutive sieves.
[b] Data taken from ASTM C 33, "Standard Specification for Concrete Aggregates." Copyright ASTM INTERNATIONAL. Reprinted with permission.

TABLE 7.5 ASTM Grading Limits for Coarse Aggregates[b]

Size Number[a]	*Nominal Max. Size*	*% Passing Each Sieve*									
		63 mm ($2\frac{1}{2}$ in.)	50 mm (2 in.)	37.5 mm ($1\frac{1}{2}$ in.)	25 mm (1 in.)	19.5 mm ($\frac{3}{4}$ in.)	12.5 mm ($\frac{1}{2}$ in.)	9.5 mm ($\frac{3}{8}$ in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)
357	50 mm (2 in.)	100	95–100	...	35–70	...	10–30	...	0–5	...	...
467	37.5 mm ($1\frac{1}{2}$ in.)	...	100	95–100	...	35–70	...	10–30	0–5	...	...
57	25 mm (1 in.)	...	...	100	95–100	...	25–60	...	0–10	0–5	...
67	19 mm ($\frac{3}{4}$ in.)	...	...	...	100	90–100	...	20–55	0–10	0–5	...
7	12.5 mm ($\frac{1}{2}$ in.)	...	...	...	...	100	90–100	40–70	0–15	0–5	...
8	9.5 mm ($\frac{3}{8}$ in.)	...	...	...	...	...	100	85–100	10–30	0–10	0–5

[a] Selected gradations. Other ASTM size numbers include 1 [90 to 37.5 mm ($3\frac{1}{2}$ to $1\frac{1}{2}$ in.)], 2 [63 to 37.5 mm ($2\frac{1}{2}$ to $1\frac{1}{2}$ in.)], 3 [50 to 25 mm (2 to 1 in.)], 4 [37.5 to 19 mm ($1\frac{1}{2}$ to $\frac{3}{4}$ in.)], 5 [25 to 12.5 mm (1 to $\frac{1}{2}$ in.)], 56 [25 to 9.5 mm (1 to $\frac{3}{8}$ in.)], and 6 [19 to 9.5 mm ($\frac{3}{4}$ to $\frac{3}{8}$ in.)].
[b] Data taken from ASTM C 33, "Standard Specification for Concrete Aggregates." Copyright ASTM INTERNATIONAL. Reprinted with permission.

where the standard sieves used are 150 μm (No. 100), 300 μm (No. 50), 600 μm (No. 30), 1.18 mm (No. 16), 2.36 mm (No. 8), and 4.75 mm (No. 4); and 9.5 mm ($\frac{3}{8}$ in.), 19 mm ($\frac{3}{4}$ in.), 37.5 mm ($1\frac{1}{2}$ in.), and larger, increasing in the size ratio 2 to 1. The fineness modulus is usually calculated only for the fine aggregate, but may be calculated for the coarse aggregate as well, based on the assumption that 100% is retained on each of the sieves from 1.18 mm (No. 16) to 150 μm (No. 100). The fineness modulus for coarse aggregate then becomes

$$FM = \frac{\Sigma\,[\text{cumulative percent retained on standard sieves, including 2.36 mm} + 400]}{100} \qquad (7.1a)$$

The fineness modulus for fine aggregate should lie between 2.3 and 3.1 (see the example in Table 7.6). A small number indicates a fine grading, whereas a large number indicates a coarse material. The fineness modulus can be used to check the constancy of grading when relatively small changes are to be expected; but it should not be used to compare the gradings of aggregates from two different sources. The parameter is quite a crude measure of grading; two aggregates with the same fineness modulus can have quite different grading curves.

The fineness modulus of fine aggregate is required for mix proportioning since sand gradation has the largest effect on workability. The finer the sand (lower fineness

TABLE 7.6 Calculation of Fineness Modulus

	Sieve Size	*Weight Retained (g)*	*Amount Retained (wt. %)*	*Cumulative Amount Retained (%)*	*Cumulative Amount Passing (%)*
Coarse aggregate	75.0 mm (3 in.)	0	0	0	100
	37.5 mm ($1\frac{1}{2}$ in.)	42	4	4	96
	19 mm ($\frac{3}{4}$ in.)	391	39	43	57
	9.5 mm ($\frac{3}{8}$ in.)	350	35	78	22
	4.75 mm (No. 4)	180	18	96	4
	2.36 mm (No. 8)	20	2	98	2
	Sample wt.	1000 g		319	
				+ 400 (from fine sieves 1.18 mm to 150 μm)	
				Σ = 719	
	Nominal maximum size = 37.5 mm ($1\frac{1}{2}$ in.)				
	Fineness modulus = 719/100 = 7.19				
Fine aggregate	4.75 mm (No. 4)	9	2	2	98
	2.36 mm (No. 8)	46	9	11	89
	1.18 mm (No. 16)	97	19	30	70
	600 μm (No. 30)	99	20	50	50
	300 μm (No. 50)	120	24	74	26
	150 μm (No. 100)	91	18	92	8
	Sample wt.	500 g		Σ = 259	
	Fineness modulus = 259/100 = 2.59				

modulus), the greater the number of particles available to improve workability. This allows an increase in the amount of coarse aggregate and a decrease in the amount of fine aggregate in the concrete. ASTM C 33 requires that the fineness modulus not vary by more than 0.2 from the value used for mix design purposes. Larger variations will cause unacceptable changes in workability and will require reproportioning. The fineness modulus of the coarse aggregate is not used for mix design purposes.

Other Grading Requirements The ASTM specifications permit a wide range of grading, particularly in the case of the fine aggregate. Other specifications may require much narrower limits, depending on whether rounded gravel and sands or angular crushed rock and manufactured sands are available. The amount of fine aggregate passing the 300 μm (No. 50) and 150 μm (No. 100) sieves affects workability, finishability, bleeding, and so on, and adequate fine material is needed for good cohesiveness and plasticity. The 10% passing the 300 μm (No. 50) sieve allowed by ASTM may not provide sufficient fines for hand-finished concrete floors, or when a smooth surface texture is wanted. In such cases, it is recommended that a fine aggregate should be used that has at least 15% passing the 300 μm (No. 50) sieve, and 3% or more passing the 150 μm (No. 100). On the other hand, ASTM C 33 allows the finer fractions to be reduced or omitted if the concrete contains workability aids: a mineral admixture, an air-entraining agent, or a high cement content. However, ASTM limits the very fine fraction, passing the 75-μm (No. 200) mesh sieve, to not more than 5% for most concrete and 3% if the concrete is subject to abrasion. The reason is that this material, often referred to as "fines," results in a very high water demand due to its high surface area and because it takes the place of cement particles on the upper surface of concrete, reducing abrasion resistance. In natural aggregates, fines are likely to be rich in clayey matter, which may increase the volume instability of the concrete.

The grading requirements for fine and coarse aggregate in ASTM C 33 do not, in themselves, guarantee that concrete will have the desired properties. As will be discussed in more detail in Chapter 10, recommendations dating back nearly 90 years address the gradation of the *combined* aggregate. The principal goal was to obtain a dense gradation, producing a low void content and a resulting minimum cement requirement. In practice, this approach cannot be used because the resulting concrete has inadequate workability. However, this does not mean that consideration of *combined* gradation is without use. For example, combined aggregate gradations can be used to optimize workability, economy, and strength, as shown in Figure 10.2 (Section 10.2).

Gap Grading Gap grading is defined as grading in which one or more intermediate size fractions are omitted (Figure 7.2d and e). Gap grading can be used to produce more economical concrete, particularly when blending of aggregates is required to obtain suitable gradings. Less sand can be used for a given workability, hence allowing the use of less cement or lower *w/c* ratios for a given slump. However, the omission of size fractions can lead to severe segregation problems, particularly in mixes of high workability. Thus, gap grading is recommended primarily for stiff mixes of very low workability that are to be compacted by vibration. Close control of mix proportions must be maintained to avoid segregation. It does not appear that gap-graded concretes are inherently superior to continuously graded concretes. Claims of improved properties can

be attributed to decreases in *w/c* ratio at a given cement content and slump. Gap-graded aggregate is often used to give better exposed aggregate finishes.

A special case of gap-grading is *no-fines concrete,* in which the fine aggregate is entirely omitted (Figure 7.2e). As would be expected, this concrete has little cohesiveness in the fresh state and cannot be compacted to a void-free condition. Consequently, no-fines concrete is a low-strength, high-permeability material in which coarse aggregate, lightly compacted by gravity or by light rodding, is cemented together by paste rather than mortar, but without completely filling the voids between the aggregate. The advantages of no-fines concrete are its low density, low drying shrinkage, and high thermal insulation, which can be used when structural requirements are not high.

Moisture Contents

Since aggregates contain some porosity,[1] water can be absorbed into the body of the particles. Also, water can be retained on the surface of the particle as a film of moisture. Thus, stockpiled aggregates can have a variable moisture content. It is necessary to have information about the moisture content, since if there is a tendency for the aggregates to absorb water, it will be removed from the paste so that the *w/c* ratio is effectively lowered and the workability of the concrete decreased. Conversely, if excess water is present at the aggregate surfaces, extra water will be added to the paste and the *w/c* ratio of the concrete will be higher than desired.

Moisture States It is convenient to define four moisture states of the aggregate as shown in Figure 7.4:

1. Oven-dry (OD). All moisture is removed from the aggregate by heating in an oven at 105°C to constant weight (overnight heating usually suffices). All pores are empty.
2. Air-dry (AD). All moisture is removed from the surface, but internal pores are partially full.
3. Saturated-surface-dry (SSD). All pores are filled with water, but there is no film of water on the surface.
4. Wet. All pores are completely filled with water with a film of water on the surface.

Of these four states, only two, the OD and SSD states, correspond to specific moisture contents, and either of these states can be used as reference states for calculating moisture contents. The AD and wet states represent the variable moisture contents that will exist in stockpiled aggregates. The SSD condition is the better choice as a reference state, for the following reasons:

1. It represents the "equilibrium" moisture state of the aggregate in concrete; that is, the aggregate will neither absorb water nor give up water to the paste.
2. The moisture content of aggregates in the field is much closer to the SSD state than the OD state.

[1]Only pores that are connected to the surface are considered in this discussion.

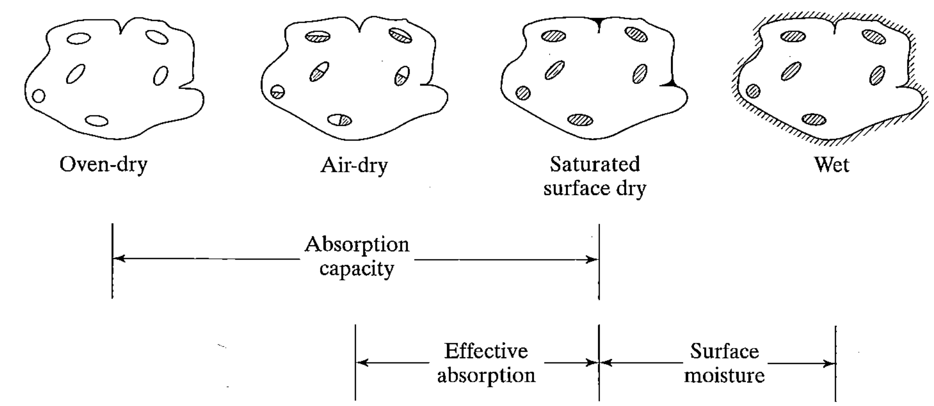


FIGURE 7.4

Moisture states of aggregates.

3. The bulk specific gravity (BSG) of aggregates is more accurately determined by the displacement method in the SSD condition.
4. The moisture content can be calculated directly from measurements of apparent BSG using the displacement method.

A major disadvantage of using the SSD state is that it is not easy to obtain a true SSD condition and it requires skill and practice to do so. Many people prefer to use the OD state as a reference point because of this. In the rest of this discussion, we use the SSD as the reference state, but the equivalent treatment using the OD state is summarized in the appendix to this chapter.

Absorption and Surface Moisture To calculate the amount of water that aggregate will add to or subtract from the paste, it is convenient to define three quantities. These are *absorption capacity, effective absorption,* and *surface moisture.*

Absorption capacity (A or absorption) represents the maximum amount of water the aggregate can absorb. It is calculated from the difference in weight between the SSD and OD states, expressed as a percentage of the OD weight:

$$A = \frac{W_{SSD} - W_{OD}}{W_{OD}} \times 100\% \tag{7.2}$$

where W_{SSD} and W_{OD} represent the weight of the aggregate sample in the SSD and OD states, respectively. The absorption capacity is used in mix-proportioning calculations and can be used to convert from the SSD to the OD system, or vice versa [$W_{SSD} = W_{OD}(1 + A/100\%)$]. Most normal-weight aggregates (fine and coarse) have absorption capacities in the range of 1 to 2%. Abnormally high absorption capacities indicate high-porosity aggregates, which may have potential durability problems. (Lightweight aggregates generally have very high absorption capacities.)

The effective absorption (EA) represents the amount of water required to bring an aggregate from the AD state to the SSD state, expressed as a fraction of the SSD weight:

$$EA = \frac{W_{SSD} - W_{AD}}{W_{SSD}} \times 100\% \quad (7.3)$$

For a required weight of SSD aggregate in a concrete mix (W_{agg}), the effective absorption can be used to calculate the weight of water absorbed (W_{abs}) to bring air-dry aggregate up to the SSD state:

$$W_{abs} = \frac{(EA)W_{agg}}{100\%} \quad (7.4)$$

Thus, the weight of air-dry aggregate batched will equal the desired weight of SSD aggregate (W_{agg}) minus W_{abs}, while the mix water will be increased by W_{abs}.

If an aggregate is close to the OD condition when batched, it takes the aggregate some time to absorb all of the water necessary to reach the SSD condition. In such cases, the effective absorption may be taken to indicate the amount of water the aggregate absorbs in 30 min. Further absorption beyond this time will be slow. This absorption, however, will be accompanied by a gradual decrease in the workability of the concrete.

The surface moisture (SM) represents water in excess of the SSD state, also expressed as a fraction of the SSD weight:

$$SM = \frac{W_{wet} - W_{SSD}}{W_{SSD}} \times 100\% \quad (7.5)$$

It is used to calculate the additional water (W_{add}) added to the concrete with the aggregate.

$$W_{add} = \frac{(SM)W_{agg}}{100\%} \quad (7.6)$$

In this case, the weight of wet aggregate will equal W_{agg} plus W_{add}, while the mix water will be decreased by W_{add} to account for the surface moisture.

Equations (7.3) through (7.6) can be put in general terms, since both the AD and wet states represent the possible conditions of stockpiled aggregates. The moisture content (MC) of the aggregate is given by

$$MC = \frac{W_{stock} - W_{SSD}}{W_{SSD}} \times 100\% \quad (7.7)$$

where W_{stock} is the weight of the aggregate in the stockpiled condition. If the moisture content is positive, it is surface moisture; if negative, it is effective absorption. Thus,

$$W_{MC} = \frac{(MC)W_{agg}}{100\%} \quad (7.8)$$

where W_{MC} is the total moisture associated with the aggregates and is positive (i.e., added) when the moisture content is positive and negative (i.e., absorbed) when the moisture content is negative.

Bulking of Sand Stockpiled coarse aggregate is generally in the AD state with an effective absorption of less than 1%. However, fine aggregate is often in the wet state, with a surface moisture typically in the range of 2 to 6%. The presence of surface moisture has a beneficial effect because it limits segregation of the fine aggregate. The reason for high surface-moisture values for the fine aggregate is that, in addition to thin surface films of moisture, additional water can be held in the interstices between fine particles as the result of formation of menisci (Figure 7.5b). The formation of these menisci creates thicker films of water between the aggregate particles, pushing them apart and increasing the apparent volume of the aggregate (Figure 7.5d). This phenomenon is known as *bulking* and can cause substantial errors in proportioning by volume. Hence, aggregate is nearly always batched by weight and measurements of unit weight are usually made on oven-dry aggregate. When sand is saturated with water, the menisci are destroyed and the volume returns to normal. Coarse aggregates show much less bulking since the particle size is large compared to the thickness of the water film and the effect of meniscus formation is slight.

Specific Gravity

The density of the aggregates is required in mix proportioning to establish weight–volume relationships. The density is expressed as the specific gravity, which is a dimensionless ratio relating the density of the aggregate to that of water:

$$SG = \frac{\text{density of solid}}{\text{density of water}} \quad (7.9)$$

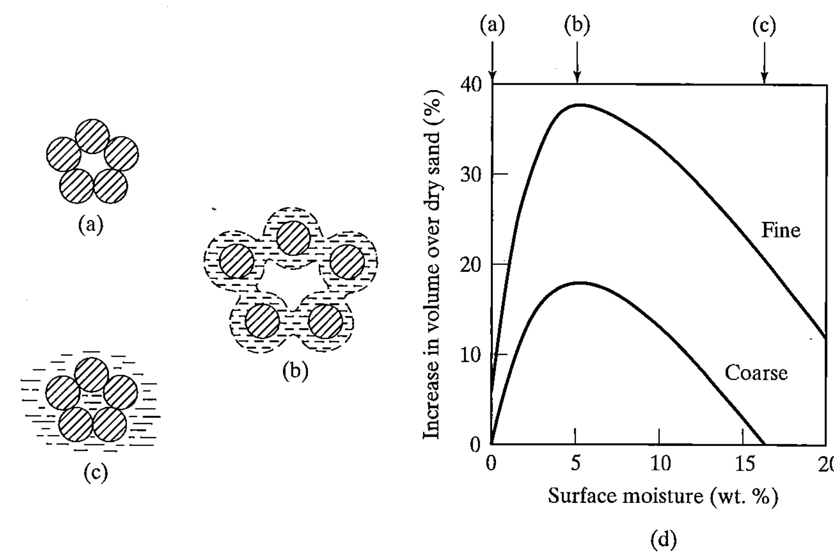


FIGURE 7.5

Bulking phenomenon of fine aggregate: (a) dry; (b) partially saturated (menisci formation); (c) fully saturated; (d) increase in volume as a function of surface moisture.

Since densities are determined by displacement in water, specific gravities are naturally and easily calculated and can be used with any system of units.

Now, it must be remembered that aggregates contain some porosity and that the specific gravity value depends on whether the pores are included in the measurement. It is also necessary to distinguish between pores that connect to the surface of the aggregate particles (permeable pores) and pores that are sealed within the particles (impermeable pores). For the purpose of the following discussion, the impermeable pores will be considered as part of the solid. Thus, we can distinguish between apparent specific gravity (ASG), which refers only to the "solid" material excluding the (permeable) pores [Eq. (7.10)], and bulk specific gravity (BSG), which includes the volume of the pores in the aggregate volume [Eq. (7.11)].

$$\text{ASG} = \frac{\text{weight of aggregate (solid only)}}{\text{volume of aggregate (solid only)}} \cdot \frac{1}{\rho_{\text{water}}} \tag{7.10}$$

$$\text{BSG} = \frac{\text{weight of aggregate (solid + pores)}}{\text{volume of aggregate (solid + pores)}} \cdot \frac{1}{\rho_{\text{water}}} \tag{7.11a}$$

$$= \frac{\text{weight of aggregate (solid + pores)}}{\text{weight of water displaced}} \tag{7.11b}$$

The BSG value is the realistic one to use since the effective volume that aggregate occupies in concrete includes the pores. It can be seen from Eq. (7.11) that the contribution of the pores to the BSG depends on the contribution of the pore medium to the weight of the aggregate. If the pores are filled with water, there is a finite contribution to weight that is absent when the pores are empty. Hence,

$$\text{ASG} > \text{BSG}_{\text{SSD}} > \text{BSG}_{\text{OD}} \tag{7.12}$$

However, since the porosity of most rocks used as concrete aggregates is only of the order of 1 to 2%, the values of all of the specific gravities are approximately the same. This is not true of lightweight aggregates, whose BSGs are strongly dependent on moisture content. The BSG of most rocks is in the range 2.5 to 2.8; a value well below this range is probably indicative of high porosity. However, the BSG of an aggregate cannot be directly related to its performance in concrete, and thus it is not a specified quantity. The only reason for specifying BSG is if a minimum density of concrete is required, since BSG directly relates to concrete density.

Determination of BSG The BSG_{SSD} of aggregates in the SSD condition is determined by the displacement method given in ASTM C 127 and C 128. The basic equation used is Eq. (7.11b). In ASTM C 127, coarse aggregate is weighed in air (W_{air}) and in water (W_{water}), and the weight of water displaced (W_{displ}) given by

$$W_{\text{displ}} = W_{\text{air}} - W_{\text{water}} \tag{7.13}$$

The weight of water displaced is, of course, numerically equal to the volume of water displaced (in the SI system), which is the same as the volume of aggregate. In

ASTM C 128, a pycnometer is weighed full of water, and then the pycnometer containing the sample of SSD sand is filled with water and weighed again. The relevant equation is

$$W_{\text{displ}} = (W_{\text{sand}} + W_{\text{pyc}}) - W^*_{\text{pyc}} \tag{7.14a}$$

where W_{sand} is the weight of the SSD sand, W_{pyc} is the weight of the pycnometer containing only water, and W^*_{pyc} is the weight of the pycnometer containing both sand and water. Equation (7.14a) can be reduced to

$$W_{\text{displ}} = W_{\text{water}} - W^*_{\text{water}} \tag{7.14b}$$

where W_{water} and W^*_{water} are the weights of water in the pycnometer in the absence and the presence, respectively, of the sand.

The displacement method cannot be used successfully using OD aggregate because absorption will take place during the measurement. Since the BSG will be different in the OD case, we write BSG_{SSD} to denote that the quantity pertains to the SSD condition.

The BSG_{OD} can be calculated using Eq. (7.11b) by substituting the oven-dry aggregate weight in the numerator. To calculate the ASG using Eq. (7.10), the weight of the aggregate is, again, the oven-dry weight, and the weight of the water displaced (solid only) is the weight of water displaced by the SSD aggregate (W_{displ}), minus the weight of the water in the pores ($W_{\text{SSD}} - W_{\text{OD}}$).

Measurement of Moisture Contents The measurement of the moisture contents of aggregates can conveniently be made using the displacement methods used to determine the BSG of the SSD aggregate. The procedure for determining the surface moisture in fine aggregate is given in ASTM C 70, but the principle behind this method can be used to determine effective absorption or surface moisture of both fine and coarse aggregate.

For a given aggregate sample from a stockpile, the following relationship holds true:

$$W_{\text{stock}} = W_{\text{SSD}} + W_{\text{MC}} \tag{7.15}$$

W_{MC} can be either positive (wet) or negative (AD). The usual measurements are then made as if the BSG of the aggregate was to be determined. (It is assumed that the aggregate reaches the SSD condition during the measurement.) Thus, for coarse aggregate using Eq. (7.11b), we get

$$\text{BSG}_{\text{SSD}} = \frac{W_{\text{SSD}}}{W_{\text{displ}}} \tag{7.16a}$$

In the experiment measuring BSG_{SSD}, the complete weight was W_{SSD}; but in this case, that is not true as Eq. (7.15) shows. However, W_{displ} can be expressed in terms of W_{SSD} as

$$\text{BSG}_{\text{SSD}} = \frac{W_{\text{SSD}}}{W_{\text{SSD}} - W^*_{\text{SSD}}} \tag{7.16b}$$

where W^*_{SSD} equals the weight of the *SSD aggregate in water.* This quantity is actually measured experimentally since the aggregate attains the SSD condition before the measurement is complete. (This is always true for wet aggregate, but could be in error if the aggregate is very dry and absorption is slow—not the usual situation.) Therefore, Eq. (7.16b) can be rearranged to obtain W_{SSD} in terms of two known quantities, BSG_{SSD} and W^*_{SSD}:

$$W_{SSD} = \frac{W^*_{SSD} \times BSG_{SSD}}{BSG_{SSD} - 1} \tag{7.17a}$$

Equation (7.17a) can now be used to solve for W_{MC} in Eq. (7.15), and W_{MC} can be expressed as a percentage of W_{SSD} (i.e., as surface moisture and effective absorption).

In the case of fine aggregate, Eq. (7.14a) can be adapted to solve for W_{displ}. Since W_{sand} is now W_{stock}, it can no longer be used in the equation, and we must write

$$W_{displ} = (W_{SSD} + W_{pyc}) - W^*_{pyc} \tag{7.17b}$$

W^*_{pyc} is measured experimentally and W_{pyc} is known from the experiment used to measure BSG_{SSD}. W_{SSD} is not known, but by substituting Eq. (7.17b) into Eq. (7.16a) and rearranging, we get

$$W_{SSD} = \frac{(W^*_{pyc} - W_{pyc})\, BSG_{SSD}}{BSG_{SSD} - 1} \tag{7.18}$$

Unit Weight

Unit weight (UW) can be defined as the weight of a given volume of graded aggregate. It is thus a density measurement and is also known as *bulk density,* but this alternative term is similar to bulk specific gravity, which is quite a different quantity, and perhaps is not a good choice. The unit weight effectively measures the volume that the graded aggregate will occupy in concrete and includes both the solid aggregate particles and the voids between them. The unit weight is simply measured by filling a container of known volume and weighing it (ASTM C 29). Clearly, however, the degree of compaction will change the amount of void space and hence the value of the unit weight. The ASTM standard method calls for compaction by rodding. (The unit weight of lightweight aggregate is often determined in a loosely packed condition without any compaction.) Since the weight of the aggregate is dependent on the moisture content of the aggregate, a constant moisture content is required. Oven-dry aggregate is used in ASTM C 29. The unit weight of the coarse aggregate is required for the volume method of mix proportioning.

The void space that must be filled with mortar can be calculated using the unit weight. The unit weight in kg/m^3 is equal to the weight of aggregate (W_a) in 1 m^3 of volume.

$$\text{total volume} = V_a + V_v = 1\,m^3 \tag{7.19}$$

Therefore,

$$V_v = (1 - V_a)\,m^3 \tag{7.20}$$

where V_a and V_v are the volume of aggregate and void space, respectively. Also,

$$V_a = \frac{W_a}{BSG \times \rho_w}\,m^3 \tag{7.21}$$

where ρ_w is the density of water in kg/m^3 and BSG is the bulk specific gravity in the dry state. Thus,

$$V_v = 1 - \frac{W_a}{BSG \times \rho_w}\,m^3 \tag{7.22}$$

or

$$V_v = 1 - \frac{UW}{BSG \times \rho_w}\,m^3 \tag{7.23}$$

or

$$\text{percent of voids} = \frac{BSG \times \rho_w - UW}{BSG \times \rho_w} \times 100 \tag{7.24}$$

The unit weights of both fine and coarse normal-weight aggregates falling within the ASTM grading limits are generally in the range of 1450 to 1750 kg/m^3 (90 to 110 lb/ft^3). However, a plot of unit weights for various aggregate blends of two particular aggregates shows a maximum unit weight when the fine aggregate content is 35 to 40% by weight of total aggregate (see Figure 7.6). From Eq. (7.22), it can be deduced that this maximum corresponds to a minimum percentage of void space (assuming that the

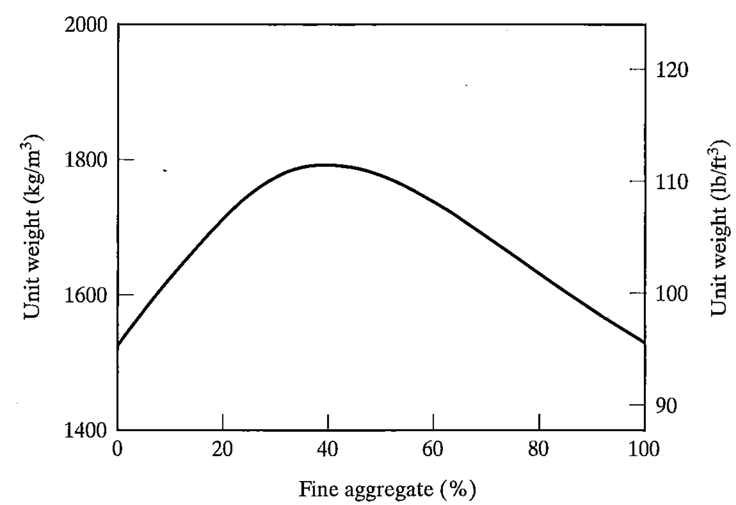


FIGURE 7.6
Variation of the unit weight of dry-rodded aggregate for blends of fine and coarse aggregate.

BSG of both aggregates is the same, which is a good approximation) that must be filled with paste. Thus, the most economical concrete is one that has close to 40% of fine aggregate in the total aggregate fraction, since the least amount of cement will be needed.

7.2 DURABILITY OF AGGREGATES

Since aggregates make up the bulk of concrete, any lack of durability of the aggregate will have disastrous consequences for the concrete. Fortunately, in most parts of North America, there are few problems concerning the durability of aggregates. In other areas, special screening tests may be required routinely to avoid problem aggregates, or special measures must be taken to counteract the effects of undesirable aggregates. The latter approach is especially important where deposits of high-quality aggregate have been worked out and more marginal material has been brought into use. Lack of aggregate durability can be conveniently divided into physical and chemical causes. The former is concerned with susceptibility of aggregates to freezing and thawing or wetting and drying, as well as physical wear. Chemical durability problems are concerned with various forms of cement-aggregate reactions.

Of the various procedures used to measure aggregate durability, field performance is usually the best. The next best choice involves tests that evaluate the aggregate when cast in concrete. However, these tests are usually more time consuming than tests performed directly on the aggregate, and therefore, a number of durability tests have been developed that can be run directly on the aggregate itself. A thorough petrographic examination can tell a lot about the potential durability of an aggregate and may eliminate the need for embarking on a lengthy test program. It is important in aggregate evaluation that proper sampling practices be followed as laid down in ASTM C 702 and D 75.

Physical Durability

Soundness Aggregates are said to be unsound if volume changes that accompany environmental changes lead to the deterioration of concrete. Volume changes can arise from alternate freezing and thawing or from repeated wetting and drying. Fortunately, rocks that undergo appreciable volume change on wetting and drying are very rare. Thus, soundness is primarily a question of freeze–thaw resistance and is the basic cause of two fairly widespread forms of concrete deterioration: surface "pop-outs" and D-cracking in pavements. The factors affecting freeze–thaw susceptibilities of aggregates are similar to those affecting hardened cement paste, which are discussed in detail in Chapter 18. Briefly, the resistance of an aggregate depends on whether high internal stresses develop when the water inside the aggregate freezes and causes a volume increase. This stress is a function of the porosity of the aggregate, its permeability, the degree of saturation, and size. A critical aggregate size can be calculated (see Chapter 18) below which freeze–thaw distress will not occur. Although for most aggregates this is greater than the normal sizes used in practice, for some poorly consolidated sedimentary rocks—cherts, graywackes, sandstones, shales, and limestones—the critical size may be less than the maximum aggregate size [in the range 12 to 25 mm ($\frac{1}{2}$ to 1 in.)].

The freeze–thaw resistance of aggregates may be evaluated in several ways. The soundness test (ASTM C 88) is supposed to simulate the effect of ice crystallization in saturated aggregate. This is done by soaking the aggregate in a saturated solution of sodium or magnesium sulfate and then drying in an oven. The weight loss occurring after a given number of cycles is determined. The crystallization of the salt in the pores is assumed to simulate the disruption of the aggregate particles by ice. There seems to be no real justification for this assumption, and testing an aggregate separately from the cement paste makes the test even less realistic. Test results exhibit high interlaboratory variability, and correlation between the test and actual service performance is not good.

An unconfined freeze–thaw test (CSA A23.2-24A) is used in Canada as an alternate to the sulfate soundness test for coarse aggregate. Samples are sieved, saturated with a 3% solution of NaCl for 24 hours, drained, and sealed in plastic jars. The jars are subjected to five freeze–thaw cycles to −18°C (0°F), and changes in the aggregate gradation are used to calculate a weighted percentage loss that correlates well with field performance. The test exhibits much less interlaboratory variability than the soundness test.

Testing the aggregate in concrete (ASTM C 666 and C 682) is the preferred approach and more widely accepted than ASTM C 88 or CSA A23.2-24A. However, difficulties in duplicating realistic temperature histories make correlations between test data and actual performance less than ideal. In spite of this drawback, good correlations have been achieved, especially for susceptibility to D-cracking. This is further discussed in Chapter 18.

Wear Resistance Wear resistance of concrete is discussed in detail in Section 17.2, but obviously the aggregate must play an important role in determining the resistance of concrete to surface abrasion and wear. A good aggregate will be hard, dense, strong, and free of soft, porous, or friable particles. The abrasion resistance of aggregates can be tested by the Los Angeles test (ASTM C 131 and C 535), which involves ball milling the aggregate with steel balls for a given time, sieving the material over a 1.70 mm (No. 12) sieve, and measuring the mass loss. The test does not correlate well with concrete wear in the field, but, rather, appears to be a better measure of the tendency of aggregates to break during handling. It is better to test the abrasion resistance of the concrete itself (see Chapter 17). Then the contribution of the paste–aggregate bond, which depends on the aggregate shape and texture, will also be taken into account.

The micro-Deval test (CSA A23.2-23A) is used in Canada as an alternate to sulfate-soundness testing for fine aggregate. (The latter appears to serve more as a measure of impact resistance for fine aggregate than as a measure of freeze–thaw resistance). The micro-Deval test involves washing to remove material finer than the 80-μm sieve, oven-drying, soaking a 500-g sample in water for 24 h, and placing the sample in a 2-L stainless steel drum with 1250 g of 9.5-mm stainless steel balls, where it is rotated at 100 rpm for 15 min. The material is again washed over a 80-μm sieve, and the loss is calculated in percent. Since the aggregate is tested in a wet condition, the test is more severe than the Los Angeles test, in which the aggregate is tested dry. The results exhibit low interlaboratory variability and, also, provide a good measure of the tendency of the fine aggregate to contribute to concrete shrinkage. A variation of the test (CSA A23.2-29A)

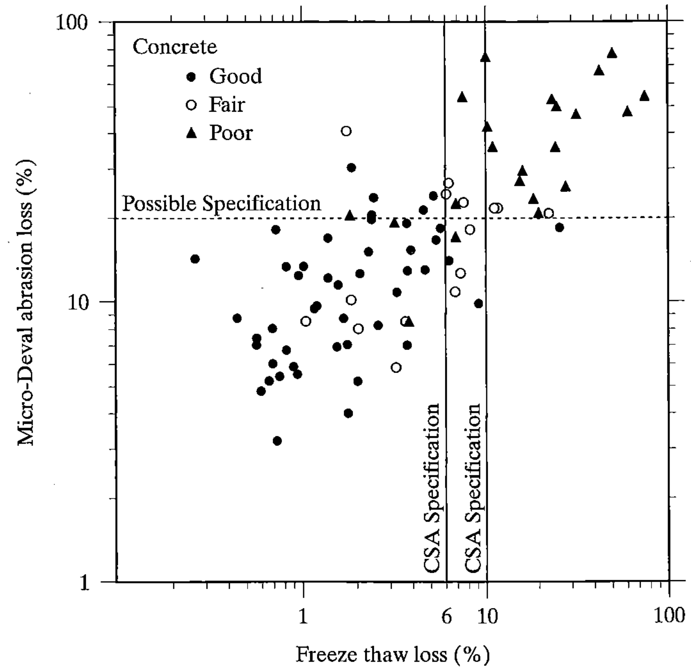


FIGURE 7.7

Comparison of field performance with micro-Deval abrasion loss and unconfined freeze–thaw loss for coarse aggregates. [From C. A. Rogers and S. A. Senior, "Recent Developments in Physical Testing of Aggregates to Ensure Durable Concretes," *Advances in Cement and Concrete*, eds. M. W. Grutzeck and S. L. Sarkar, American Society of Civil Engineers, New York, pp. 338–361 (1994).]

is used for coarse aggregate, with a 1500-g sample, a 95 or 105-min run time (depending on the maximum size of the aggregate), and a 1.25-mm sieve. Results of the micro-Deval test and the unconfined freeze–thaw test (CSA A23.2-24A) are compared to concrete field performance in Figure 7.7.

Chemical Resistance

Most chemical-durability problems result from a reaction between reactive silica in aggregates and alkalis contained in the cement. The most familiar problem is the *alkali–aggregate reaction* (also referred to as the *alkali–silica reaction*), but other less-common reactions have been identified in recent years, and these will also be discussed. Occasionally, other kinds of chemical distress occur. For example, iron pyrites (FeS) may react expansively in the presence of calcium hydroxide to form ferrous sulfate and ultimately ferric hydroxide. This reaction can cause popouts and staining. Natural gypsum will cause sulfate attack if present in significant amounts. Small quantities of zinc or lead are occasionally found in aggregate deposits and may greatly delay setting and early hardening of the concrete.

Alkali–Aggregate Reaction In the United States, there were many reported failures of concrete structures built during the late 1920s to the early 1940s. These failures were the result of overall cracking throughout the structure manifested at the surface as extensive *map cracking* or *pattern cracking* (see Figure 7.8), frequently accompanied by gel exuding from the cracks or surface popouts and spalling. Such problems were confined mostly to the west and southwest of the country, as well as Kansas, Nebraska, Alabama, and Georgia.

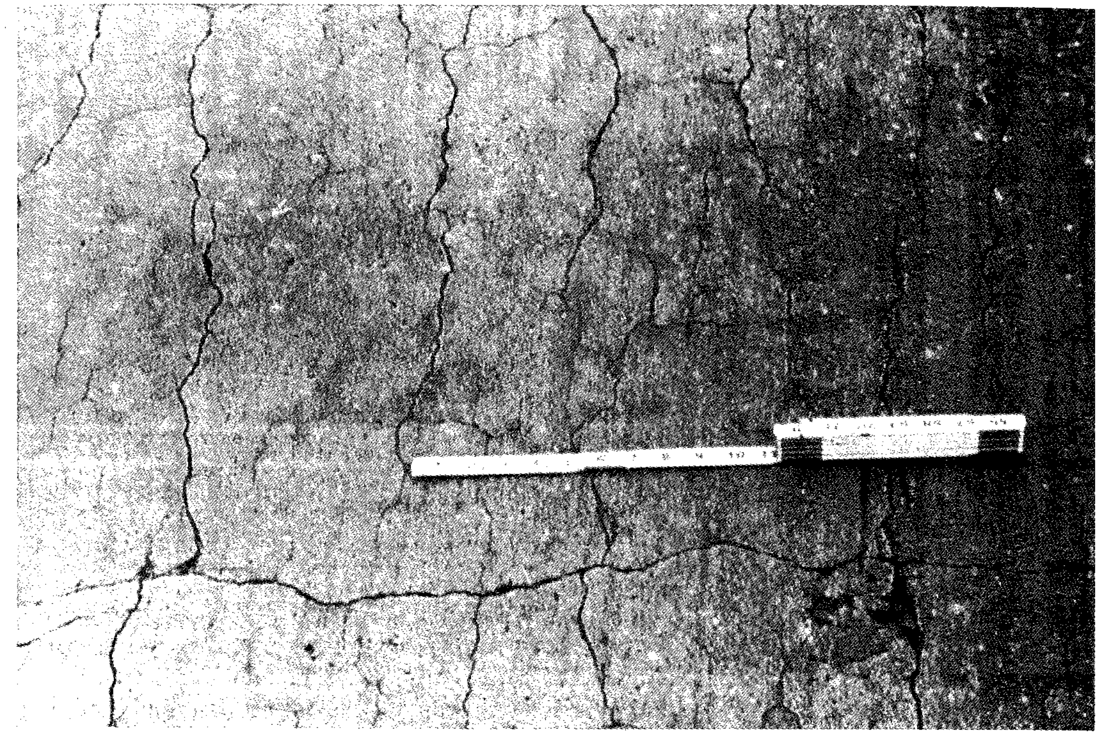


FIGURE 7.8

Example of well-developed map cracking caused by alkali–aggregate reaction. (Photograph courtesy of Portland Cement Association.)

Classic research by Stanton[2] in the early 1940s correctly diagnosed the failures as being due to expansions caused by a chemical reaction between the alkalis contained in the cement paste and certain reactive forms of silica within the aggregate (Table 7.7). Stanton found opal and chert to be common forms of reactive silica, but the list is considerably broader, and naturally occurring volcanic glasses are a widespread form of reactive silica. It appears that various forms of silica have differing reactivities, depending on the degree of crystallinity, internal porosity, crystallite size, and internal crystal strain. Opal is the most reactive form of natural silica, being both amorphous and porous. Container glass (borosilicate) containing sodium or potassium is also a very reactive material. Over time, it has become clear that different aggregates react at different rates and that damage due to the alkali–aggregate reaction may not be apparent for many years after a structure is put into service. Most of the structural failures in the 1930s and 1940s occurred within 1 to 10 years, probably involving opaline

[2]T. E. Stanton, *Proceedings ASCE*, Vol. 66, pp. 1781–1811 (1940).

TABLE 7.7 Forms of Reactive Silica in Rocks That Can Participate in Alkali–Aggregate Reaction

Reactive Component	*Physical Form*	*Rock Types in Which It Is Found*	*Occurrence*
Opal	Amorphous	Siliceous (opaline limestones, cherts, shales, flints)	Widespread
Silica glass	Amorphous	Volcanic glasses (rhyolite, andesite, dacite) and tuffs; synthetic glasses	Regions of volcanic origin; river gravels originating in volcanic areas; container glass
Chalcedony	Poorly crystallized quartz	Siliceous limestones and sandstones, cherts and flints	Widespread
Cristobalite tridymite	Crystalline	Opaline rocks, fired ceramics	Uncommon
Quartz	Crystalline	Quartzite, sands, sandstones, many igneous and metamorphic rocks (e.g., granites and schists)	Common, but reactive only if highly strained or microcrystalline

rocks. However, in many cases diagnosed since then, the severe deterioration has not occurred until after 15 to 40 years, indicating conditions of lower reactivity. In the case of the Hurdman Bridge in Canada, the problem was not documented until 50 years after the bridge was constructed (the bridge remained in service for another 30 years).

Factors affecting expansion. Stanton was able to determine the factors that control the alkali–aggregate expansion: (1) nature of reactive silica, (2) amount of reactive silica, (3) particle size of reactive material, (4) amount of available alkali, and (5) amount of available moisture. We have already discussed the nature of the reactive silica, but the amount is also important. Reference to Figure 7.9 shows that maximum expansion occurs at a particular reactive silica content (the "pessimum percentage"), with decreasing expansion at higher percentages. The pessimum (opposite of optimum) percentage depends on the form of the reactive silica, the degree of alkalinity, and the *w/c* ratio. The pessimum percentage may range from 2 to 10%, but can be as high as 100%. The particle size of the reactive material is also an important factor (Figure 7.10), with expansion increasing as particle size decreases. The alkali content of the cement, and hence of the concrete, is another important factor. Stanton established experimentally that below 0.6%, Na_2O equivalent[3] deleterious expansions usually do not occur (see Figure 7.11). More recent studies indicate that both the ratio of reactive silica to alkali (Figure 7.12) and the total alkali content of the concrete on a mass percent volume basis (Figure 7.13) play important roles in the nature of the reaction products. As shown in Figure 7.13, the upper limit on the Na_2O equivalent (from all sources) for a safe condition appears to be 4 kg/m^3 (6.8 lb/yd^3). The exact form of the

[3]The Na_2O equivalent is given by $Na_2O + 0.658K_2O$. The factor 0.658 merely accounts for the difference in the molecular weights of Na_2O and K_2O. Thus, $0.658K_2O$ represents the weight of the number of moles of Na_2O that is equivalent to the weight of K_2O.

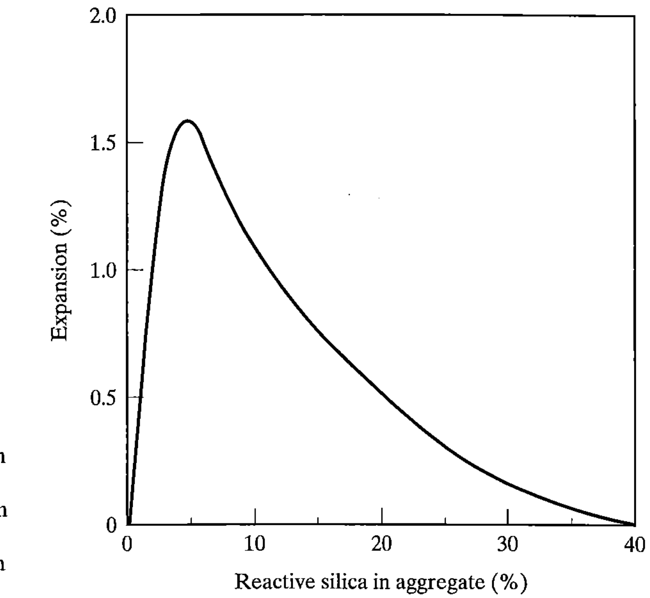


FIGURE 7.9

Effect of the content of reactive silica in an aggregate on the expansion of concrete due to the alkali–aggregate reaction. [From H. Woods, *Durability of Concrete Construction*, Monograph No. 4., American Concrete Institute, Detroit, Mich (1968).]

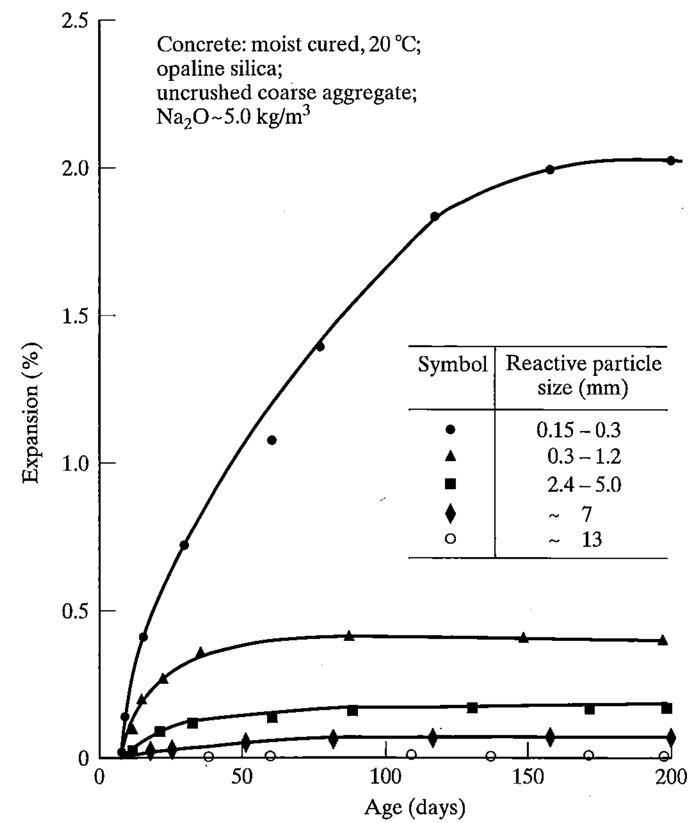


FIGURE 7.10

Effect of reactive particle size on the relationship between expansion and age: *w/c* ratio = 0.41, aggregate/cement ratio = 3. [From D. W. Hobbs, *Alkali-Silica Reaction in Concrete*, Thomas Telford, London (1988).

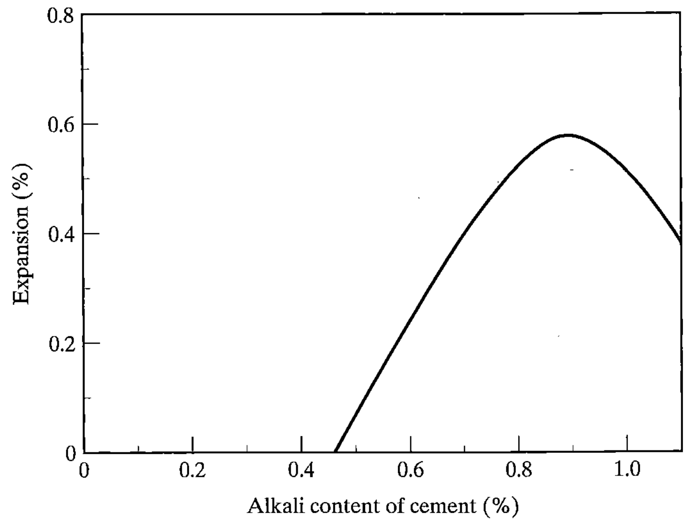


FIGURE 7.11

Effect of the alkali content of the cement on alkali–aggregate expansion. [From H. Woods, *Durability of Concrete Construction*, Monograph No. 4., American Concrete Institute, Detroit, Mich. (1968).

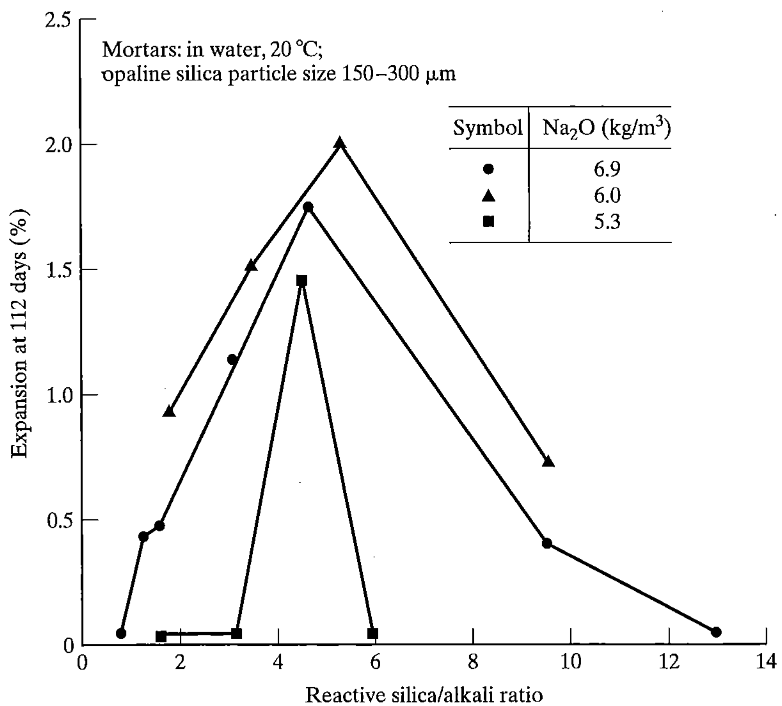


FIGURE 7.12

Dependence of expansion on reactive silica/alkali ratio: *w/c* ratio = 0.41, aggregate/cement ratio = 2. [From D. W. Hobbs, *Alkali-Silica Reaction in Concrete*, Thomas Telford, London, (1988).]

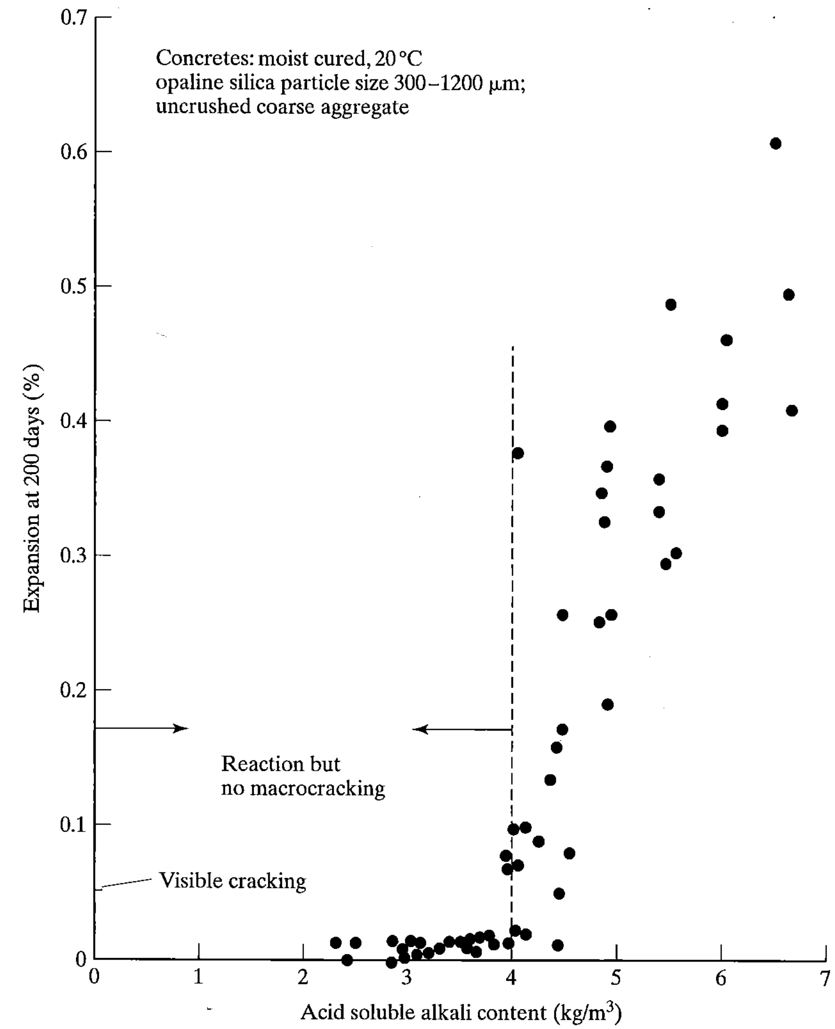


FIGURE 7.13

Expansion at 200 days as a function of acid soluble alkali content. [From D. W. Hobbs, *Alkali-Silica Reaction in Concrete*, Thomas Telford, London (1988).]

curves in Figures 7.9 to 7.12 depends on the physical and chemical nature of the reactive component. Finally, there must be sufficient moisture available within the concrete for the alkali–aggregate reaction to proceed.

Mechanism of alkali–aggregate reaction. The experimental data given previously have been used as the basis of practical control of the alkali–silica reaction, as will be further discussed shortly, but have not provided a detailed explanation for the cause of distress. The following explanation is based on a synthesis of a broad range of

research by Helmuth and Stark[4] in which they concluded that the basic mechanisms are described by an earlier model developed by Powers and Steinour.[5]

Helmuth and Stark observed that the alkali–silica reaction results in the production of two-component gels—one component is a nonswelling calcium–alkali–silicate–hydrate [C-N(K)-S-H] and the other is a swelling alkali-silicate–hydrate [N(K)-S-H]. When the alkali–silica reaction occurs in concrete, some nonswelling C-N(K)-S-H is always formed. The reaction will be safe if this is the only reaction product, but unsafe if both gels form. The key factor appears to be the relative amounts of alkali and reactive silica. The overall process proceeds in a series of overlapping steps:

1. In the presence of a pore solution consisting of H_2O and Na^+, K^+, Ca^{2+}, OH^- and $H_3SiO_4^-$ ions (the latter from dissolved silica), the reactive silica undergoes depolymerization, dissolution, and swelling. The swelling can cause damage to the concrete, but the most significant volume change results from cracking caused by subsequent expansion of reaction products.

2. The alkali and calcium ions diffuse into the swollen aggregate resulting in the formation of a nonswelling C-N(K)-S-H gel, which can be considered as C-S-H containing some alkali. The calcium content depends on the alkali concentration, since the solubility of CH is inversely proportional to the alkali concentration.

3. The pore solution diffuses through the rather porous layer of this C-N(K)-S-H gel to the silica. Depending on the relative concentration of alkali and the rate of diffusion, the result can be safe or unsafe. If CaO constitutes 53% or more of the C-N(K)-S-H on an anhydrous (without water) weight basis of the gel, only a nonswelling gel will form. For high-alkali concentrations, however, the solubility of CH is depressed, resulting in the formation of some swelling N(K)-S-H gel that contains little or no calcium. The N(K)-S-H gel by itself has a very low viscosity and could easily diffuse away from the aggregate. However, the presence of the C-N(K)-S-H results in the formation of a composite gel with greatly increased viscosity and decreased porosity.

4. The N(K)-S-H gel attracts water due to osmosis, which results in an increase in volume, local tensile stresses in the concrete, and eventual cracking. Over time, the cracks fill with reaction product, which gradually flows under pressure from the point of its initial formation.

The first step depends on the alkalinity of the solution and does not involve soluble alkali ions per se, although they control the alkalinity [i.e., (OH^-)] of the solution. The higher the alkalinity of the cement, the greater the solubility of amorphous silica and the rate at which it dissolves. Crystalline silica will also be affected by the pH of the surrounding medium, but less dramatically. The initial porosity of the aggregate will

[4] R. Helmuth and D. Stark, "Alkali-Silica Reactivity Mechanisms," *Materials Science of Concrete III*, ed. J. Skalny, American Ceramics Society, Westerville, Ohio, pp. 131–208 (1992).
[5] T. C. Powers and H. H. Steinour, *Journal of the American Concrete Institute*, Proc. Vol. 51, pp. 497–516, 785–811 (1955).

also determine the rate and extent of the initial step and whether it can take place throughout the particle or initially only on the surface.

Calcium ions are always available to some degree in hydrated cement paste, and CH tends to form preferentially at the surface of aggregate particles. Therefore, Ca^{2+} ions are available to form the C-N(K)-S-H component of the gel in Steps 2 and 3. Whether C-N(K)-S-H represents the sole component depends on the relative amounts of silica and alkali. For low S/N ratios, the pH of the pore solution remains high and the solubility of calcium remains low, resulting in formation of N(K)-S-H. For slightly higher S/N ratios, the increase in reactive silica results in an increase in the total reaction product. For still higher S/N ratios, however, a greater portion of the alkali is tied up, decreasing the pH of the pore solution and increasing both the amount of calcium in solution and the nonswelling C-N(K)-S-H component of the gel. This explains the occurrence of the "pessimum percentage," as shown in Figure 7.9.

The gel that is formed in Step 3 in a highly alkaline environment is much denser (often described as massive) than that formed at a lower pH. This denser gel likely plays a role in preventing the dissolved hydrous silica from leaving the vicinity of the aggregate particles. The osmotic pressure developed in Step 4 is high enough to crack concrete. Interestingly, the alkali–silica reaction is slowed or stopped as compressive stress is applied to concrete.

Accelerated tests have shown that alkalis, in the form of salts, can accelerate the alkali–silica reaction. On the other hand, it has long been demonstrated that pozzolans (reactive silicas, see Chapter 5) reduce the severity of the reaction, if used in adequate quantities. Pozzolans, however, will be ineffective under conditions of unlimited availability of alkalis (such as in structures in salt water). Very small silica particles (below the sizes shown in Figure 7.10) can encourage a rapid reaction without deleterious effects. In this case, the reaction moves rapidly through Step 4 and the reaction products are well distributed, resulting in little effect of gel viscosity and a uniform distribution of reaction products (decreasing local concentration gradients and subsequent osmotic pressures).

Control of alkali–aggregate reaction. Knowledge of the factors affecting the alkali–aggregate reaction can be used to control its effects in concrete. The following approaches can be used: (1) control of pH in the pore solution, (2) control of alkali concentrations, (3) control of amount of reactive silica, (4) control of moisture, and (5) alteration of alkali–silica gel.

Mineral admixtures (fly ash, ground-granulated blast furnace slag, and silica fume) have been used to reduce the effects of the alkali–aggregate reaction (see, e.g., Figure 7.14a). The beneficial effects of these materials can be tied to (1) a dilution of the cement due to partial replacement with the mineral admixture, (2) a reduced pH of the pore solution, (3) the resulting increase in solubility of calcium, and (4) the subsequent formation of nonexpanding C-N(K)-S-H in place of swelling N(K)-S-H in Step 3. It has been observed that it takes a large quantity of pozzolan or slag to successfully control the reaction. Typically, adequate protection is attained by replacing 15 to 20% of the cement by a Class F fly ash. Using a high lime, Class C fly ash requires a considerably greater replacement, typically 35 to 40%. The reason for the lower efficiency of the Class C fly ash is likely due to the greater portion of the silica in the fly ash that is tied up by the lime, lowering the amount of silica available to control the alkali–silica

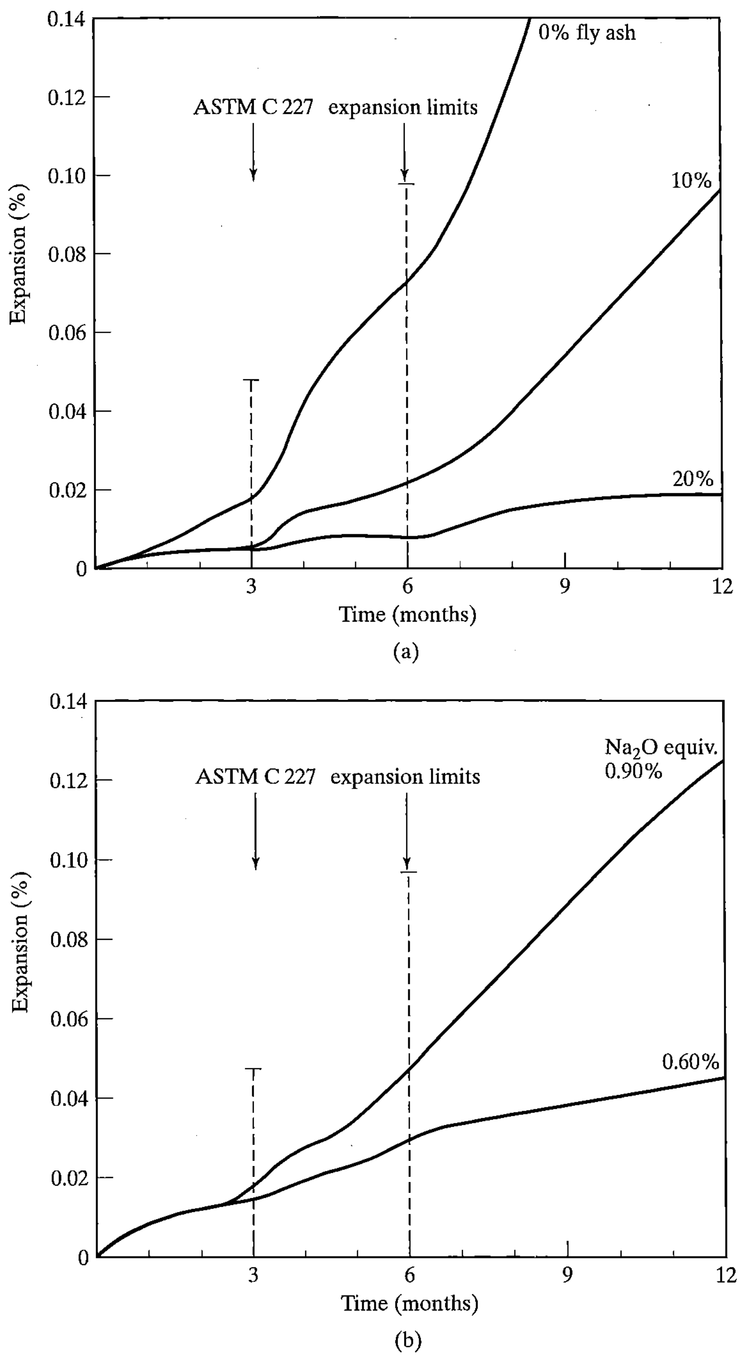


FIGURE 7.14

Effect of (a) pozzolan additions and (b) alkali content of the cement on the progress of the alkali–aggregate reaction (different aggregate used in each case). [From D. Stark, in *Proceedings, Fourth International Conference on the Effects of Alkalis in Cement and Concrete*, Purdue University, W. Lafayette, Ind. (1978).]

reaction. Because of its lower efficiency, some authorities prohibit the use of Class C fly ash in concrete-containing reactive aggregates. Alkalis in fly ash will also reduce its effectiveness. Silica fume is highly effective in controlling the alkali–aggregate reaction due to its high silica content and high surface area, allowing for a rapid reaction that results in the formation of C-N(K)-S-H while the concrete is in a fresh state; cement replacement values of 10 to 15% are typical. Under any conditions, it is important to use enough pozzolan, since low quantities of added reactive silica may increase, rather than decrease, the severity of the alkali–silica reaction (Figure 7.9). When slag is used, replacement quantities of 50% or more are common. Prior to use, mineral admixtures should be tested with the other concrete ingredients to establish their viability in controlling the alkali–aggregate reaction.

Alkali concentrations can be controlled by using low-alkali cements with less than 0.6 wt.% Na_2O equivalent (see Figure 7.14b). However, over the years, the alkali contents of cements have risen due to changes in manufacturing technology and environmental regulations. Thus, in addition to low-alkali cements, blended cements such as Types IP and IS should be considered as feasible means of controlling the alkali–aggregate reaction. Higher alkali contents in cement, together with the increasing use of marginal aggregates, are likely to make the alkali–silica reaction a continuing problem area.

Damage due to the alkali–aggregate reaction has been observed in some concretes in which low-alkali cements have been used, and it has been rightly pointed out that the alkali content of the cement may not be the appropriate parameter and that the total alkali content of the concrete is of much greater import. The alkalis can come, not only from cement, but also from chemical and mineral admixtures and from the aggregate itself. An alkali-rich basalt in New Zealand[6] has been shown to provide alkalis that initiate the alkali–aggregate reaction, even when low-alkali cement is used. Concretes with high cement factors can be expected to exhibit more problems than concretes with lower cement factors. For this reason, 3 kg/m^3 (5 lb/yd^3) represents a safe, practical upper limit for total alkalis when reactive aggregates are used [below the value of 4 kg/m^3 (6.8 lb/yd^3) shown in Figure 7.13].

The best solution, whenever it is practical, is the avoidance of a susceptible aggregate based on petrographic analyses and service records. River gravels are often the most dangerous aggregates because they may contain relatively small amounts of the reactive particles. A careful petrographic examination followed by beneficiation of the aggregate can enable the producer to control the amount and particle size of the reactive component to bring expansion within safe levels.

A low *w/c* concrete is very impermeable and may also help to limit the supply of water needed to cause the alkali–silica gel to swell, but will only slow the reaction. No adverse expansion will occur when external moisture is not available. Lithium and barium salts have been used as additives to control alkali–aggregate expansions, and the effectiveness of these additions appears to be based on the preferential formation of non-swelling lithium and barium silicate hydrates. Lithium-containing glasses are known not to cause expansion in mortars.

[6] R. Goguel and N. B. Milestone, "Auckland Basalts as a Source of Alkali in Concrete," *Proc.*, Third CANMET/ACI Industrial Conference on Advances in Concrete Technology, ACI SP-171, American Concrete Institute, Farmington Hills, MI, pp. 429–443 (1997).

Evaluation of aggregates. Observing field performance is the best method for evaluating the potential reactivity of an aggregate. However, when long-term data are not available, short-term laboratory tests can be used to indicate the potential reactivity of an aggregate, although the results often do not predict field behavior accurately. ASTM C 289 is a quick chemical test that measures the solubility of silica when powdered aggregate is treated with sodium hydroxide. The results are used to assess reactivity. Certain minerals may interfere with the test, and petrographic examination should be done to evaluate the results. Even so, the test is not a reliable indicator of alkali reactivity. A more accurate test involves the deleterious expansion in a mortar bar (ASTM C 227). In this test, coarse aggregate is reduced to a fine grading by crushing and used to cast standard mortar bars ($25 \times 25 \times 285$ mm or $1 \times 1 \times 11\frac{1}{2}$ in.) using the cement that will be used on the job. The bars are stored in a moist condition at 38°C (100°F) to accelerate the alkali–aggregate reaction. Expansion should not exceed 0.05% after three months or 0.1% after six months. This is not always a reliable test since deleterious expansions can occur after six months (see Figure 7.14), and the aggregate content in the bars may differ significantly from the pessimum percentage. The method has proved to be particularly unreliable for aggregates containing microcrystalline or strained quartz.

ASTM C 1293 and CSA A23.2-14A present a longer term test in which the alkali content of the cement is artificially increased to 1.25% at the time of fabrication of concrete prisms that are 75×75 mm (3×3 in.) in cross section by 275 to 405 mm (11 to 16 in.) long. The prisms are stored at 100% relative humidity at 38°C (100°F). Nonreactive fine aggregate must be used when coarse aggregate is evaluated, and nonreactive coarse aggregate must be used when fine aggregate is evaluated. Expansion must be less than 0.04% at one (ASTM C 1293) or two years (CSA A23.2-28A). The greater realism of the test and the longer test period makes this test method generally applicable for all types of aggregates.

ASTM C 1260 (CSA A23.2-25) is an accelerated test method that also has an enhanced alkali exposure. The expansion of mortar bars stored in a 1-N sodium hydroxide solution at 80°C (176°F) is measured at 1, 3, 7, 10, and 14 days. The NaOH solution simulates a high alkali content. Expansion should not exceed 0.10% after 14 days; expansions over 0.20% indicate problematic aggregates, while aggregates that show expansions between 0.10 and 0.20% are found to have variable field performance. The method is fast and effective in identifying a wide range of reactive aggregates, including aggregates that have not been identified by ASTM C 227.

A mortar bar test, using Pyrex glass as aggregate, is the basis of a test (ASTM C 441) to evaluate the effectiveness of mineral admixtures in controlling the alkali–aggregate reaction. To be considered effective, expansions should be reduced to 25% (slag) or 20% (silica fume) of those measured in mortar bars made with high-alkali cement without the pozzolan. For slag, when the job cement and ratio of cement to slag are known, the criterion becomes a maximum expansion of 0.02% at 14 days. For fly ash and natural pozzolans to be considered effective, expansion must not exceed that obtained using control specimens containing low-alkali cement. Because Pyrex glass contains alkalis, and so represents a less realistic situation, some researchers prefer to use highly reactive natural opals as the reference aggregate.

Other Alkali–Silica Reactions Aggregate deposits along some river systems in Kansas, Nebraska, Iowa, Missouri, and Wyoming are susceptible to an obscure cement–aggregate reaction. These materials, known as *sand–gravel* aggregates because of their grading, are highly siliceous and cause map cracking in concrete. This distress is reduced but not prevented by the use of pozzolans or low-alkali cements. Replacement of 30% of the aggregate with (nonreactive) crushed limestone appears to be an effective remedy, perhaps because it reduces the available reactive silica (Figure 7.9). It is believed that the reaction is not the result of a single cause. The alkali–aggregate reaction may be a major factor in many cases, especially when alkalis are concentrated near the concrete surface due to evaporation, but drying shrinkage and freezing and thawing are also involved.

The mortar bar test is used in a modified form for evaluation of sand–gravel reactivity (ASTM C 342). Storage is at 55°C (131°F), and a drying period is also included. More reliable results, however, appear to be obtained by combining petrographic examination (ASTM C 295) and the standard mortar bar test (ASTM C 227) with field records to establish potential reactivity problems.

Another kind of alkali–silica reaction has been identified in structures in Nova Scotia and other parts of eastern Canada as well as in other countries. The rock types involved are graywackes, argillites, phyllites, siltstones, and so on—sedimentary rocks composed largely of clay minerals. It appears that the primary cause of attack is an *alkali–silicate reaction* (clays are layer-lattice silicates), which causes "exfoliation" of the clay minerals. This can be best described as a separation of flat, platey particles of the clay minerals, which are normally compacted tightly together. This reaction takes place quite slowly, so that the effects of the alkali–silicate reaction may not be apparent for many years, but it appears that the expansions may occur indefinitely. Some normal alkali–aggregate reactions may occur simultaneously because microcrystalline quartz is often present in these rocks.

Alkali–Carbonate Reaction Expansive reactions involving carbonate rocks have occurred in some midwestern and eastern states and in eastern Canada. The rocks in question are dolomitic limestones [$CaMg(CO_3)_2$] containing some clay, but not all reactive rocks in this category cause deleterious expansions. It is known that expansive rocks have the following features: (1) very fine grained dolomite (small crystals), (2) considerable amounts of fine-grained calcite, (3) abundant interstitial clay, and (4) the dolomite and calcite crystals evenly dispersed in a clay matrix. The reaction is not properly understood. It has been established that dedolomitization occurs according to the reaction

$$\underset{\text{dolomite}}{CM\bar{C}_2} + \underset{\text{(from paste)}}{NH \text{ (or KH)}} \rightarrow \underset{\text{brucite}}{MH} + \underset{\text{calcite}}{C\bar{C}} + \underset{\text{soluble carbonates}}{N\bar{C}} \tag{7.25}$$

The alkali carbonate, $N\bar{C}$, later reacts with calcium hydroxide to regenerate the alkali hydroxide, which is again available to participate in the dedolomitization reaction.

$$N\bar{C} + CH \rightarrow NH + C\bar{C} \tag{7.26}$$

The reaction shown in Eq. (7.25) is expansive, and the rate at which it proceeds depends on the crystal size and concentration of the accompanying calcite. Dedolomitization is fastest when the calcite/dolomite ratio is about 1, the crystal size is less than 40 μm, and the clay content is between 5 and 25%. Water must be present for expansion to occur and large aggregate particles tend to cause more damage than small particles. The exact role of the clay is unclear; it may be that its role is primarily a question of structural weakening of the carbonate skeleton so that expansion caused by dedolomitization can cause internal cracking. Another suggestion is that dedolomitization exposes clay in a special "active" state that can react with alkali metal ions to form a swelling clay. However, it is also possible that the clay acts as a semipermeable membrane, thereby setting up an osmotic cell involving the soluble alkali carbonates produced on dedolomitization. The critical amount of clay is dependent on the amount of dolomite in the carbonate fraction.

The alkali–carbonate reaction can be controlled by keeping the alkali content of the cement low or by diluting the reactive aggregate with less-susceptible material. Unlike the alkali–aggregate (silica) reaction, there is no pessimum percentage of reactive aggregate, but it appears that the alkali content of the cement should be kept as low as 0.40% for adequate protection. Alkalis from deicing chemicals containing sodium chloride, however, can supply NH to the dedolomitization reaction and thus aggravate the problem even when a low-alkali cement is used. Pozzolans are not effective in controlling the alkali–carbonate reaction.

Field service records provide the best measure of the reactivity of carbonate rocks. When such records are not available, potential reactivity can be determined by measuring the expansion of rock cylinders stored in sodium hydroxide solution (ASTM C 586) or concrete prisms containing the questionable aggregate stored at 100% relative humidity and 23°C (73.4°F) (ASTM C 1105). Expansions of the rock cylinders greater than 0.10% warrant further testing in concrete. Expansions of the concrete prisms greater than 0.015% at three months, 0.025% at six months, or 0.030% at one year are considered deleterious when job cement is used (ASTM C 1105).

Deleterious Substances For satisfactory performance, concrete aggregates should be free of deleterious substances. ASTM C 33 establishes limits (see Table 7.8) that depend on the type of exposure and the application. We can divide these into two categories —impurities and unsound particles.

Impurities. Impurities can be classified as solid materials or soluble substances. Solid materials are generally present in a very finely divided state, passing the 75-µm (No. 200) sieve. Such material will appreciably increase the water requirements for workable concrete and reduce the abrasion resistance of hardened concrete if present in large amounts. The fine fraction is also likely to adhere to the surfaces of the large aggregate particles, isolating those particles from the surrounding concrete and causing a reduction in strength. Materials in this class are commonly silt, rock dust, and organic matter. Organic matter may also react chemically with the alkaline cement paste, forming soluble organic compounds that interfere with the setting processes. If sea-dredged aggregates are used, they must be thoroughly washed to avoid serious problems arising from salt contamination. Care should be taken during transportation and

TABLE 7.8 Limits for Deleterious Substances in Concrete Aggregates[a]

	Maximum Allowable (%)			
	Coarse Aggregate		*Fine Aggregate*	
Material	*Flatwork*[b]	*Structural*	*Flatwork*[b]	*Structural*
Clay lumps and friable particles	3.0	5.0[c]	3.0	3.0
Coal and lignite	0.5	0.5	0.5	1.0
Chert	5.0	5.0[d]	–	–
Material finer than 75-μm (No. 200) sieve	1.0	1.0	3.0	5.0

[a]These figures apply to concrete subjected to severe weathering; limits can be increased in moderate climates or for interior concrete. Data taken from ASTM C 33, "Standard Specification for Concrete Aggregates." Copyright ASTM INTERNATIONAL. Reprinted with permission.
[b]Concrete subject to abrasion and wear or frequent wetting (e.g., pavements, bridge decks, curbs, etc.).
[c]2.0 for architectural concrete.
[d]3.0 for architectural concrete.

storage of aggregates to ensure that inadvertent chemical contamination does not occur.

Unsound particles. A variety of unsound particles can occur in small quantities in aggregates. Soft particles such as clay lumps, wood, and coal will cause pitting and scaling at the surface. Coal may also swell in the presence of moisture or release undesirable organic compounds that interfere with setting and hardening. Weak, friable particles of low density, such as many shales and pumice, should also be avoided if a good wearing surface is needed. Reactive materials such as sulfides, gypsum, and cherts can also lead to problems, as discussed earlier.

7.3 NONSTANDARD AGGREGATES

Aggregate Beneficiation

If an aggregate does not meet the specifications required for the job, after the appropriate ASTM tests have been conducted, the engineer is faced with two courses of action. Either the aggregate can be rejected, or measures can be taken to bring the aggregate up to specifications. Beneficiation of aggregate may be the only available solution if aggregate supplies are scarce, but this will add to its cost and hence that of the concrete. Some possible treatments are summarized in Table 7.9.

Soft, friable particles can be removed by crushing, although this process will also remove some sound material. Soft, lightweight particles can be removed by various forms of separation based on the lower specific gravities of these deleterious substances. Heavy-media separation allows removal of light particles, which float to the top of the liquid. The density of particles removed can be controlled by varying the

TABLE 7.9 Beneficiation of Aggregates

Treatment	*Removal*
Crushing	Friable particles
Heavy-media separation	Lightweight particles
Reverse air or water flow	Lightweight particles
Hydraulic jigging	Lightweight particles
Elastic fractionation	Lightweight particles
Washing and scrubbing	Surface coatings, finely divided materials, organics
Selective quarrying, crushing, and blending	Control or removal of deleterious components

specific gravity of the liquid. This is a useful method for improving heavyweight aggregates. Reverse water flow or air flow can be used to remove wood and other lightweight material from coarse aggregate. Hydraulic jigging or vertical pulsation of water through a horizontal bed of aggregate stratifies the material in layers of increasing density with the lightest materials at the top. Elastic fractionation is a dry method of density separation. Aggregate is allowed to fall on an inclined steel plate, and the rebound of each particle depends on its elastic modulus. Hard, dense particles with a high modulus will rebound farther than will soft, porous particles with a low modulus. Thus, a suitable location of collection bins can provide a good separation. These methods will remove such material as wood, coal, clay lumps, sandstones, cherts, shale, chalk, and the like. Finely divided material or surface coatings may also be removed, but special washing and scrubbing procedures must be used.

Selective crushing and blending of the aggregate may be required to provide the desired size gradation. If chemical reactivity is suspected, aggregate composition should be carefully monitored. It may be possible to avoid deleterious aggregates by selective quarrying or to obtain safe concentrations or size ranges of deleterious substances by controlled blending and crushing.

Waste Materials as Aggregates

The possibility of using solid wastes as aggregate in concrete serves as one promising solution to the escalating solid-waste problem. The use of solid wastes is not a new concept: Industrial wastes are the basis of many concrete admixtures; fly ash has been used as a pozzolanic material in concrete for many decades; slags have been used both as aggregate and as a cementitious material; and cans have been shredded to provide steel fibers for reinforcement. The use of concrete for the disposal of solid wastes has concentrated mostly on aggregates, since they provide the only real potential for using large quantities of waste materials.

A wide variety of materials come under the general heading of solid wastes (see Table 7.10). These range from municipal and household garbage, or building rubble, such as brick and concrete, through unwanted industrial byproducts such as slag and fly ash or discarded unused materials such as mine tailings.

When considering a waste material as a concrete aggregate, three major areas are relevant: (1) economy, (2) compatibility with other materials, and (3) concrete properties. The economical use of a waste material depends on the quantity available, the

TABLE 7.10 Typical Solid Wastes that have been Considered as Aggregate for Concrete

Material	*Composition*	*Industry*	*Annual Amount (10^6 tons)*
Mineral wastes	Natural rocks	Mining and mineral processings	2000
Blast-furnace slags	Silicates or aluminosilicates of calcium and magnesium silicate glasses	Iron and steel	30
Metallurgical slags	Silicates, aluminosilicates, and glasses	Metal refining	30
Bottom ash	Silica glasses	Electric power	15
Fly ash	Silica glasses	Electric power	60
Municipal wastes	Paper, glass, plastics, metals	Commercial and household wastes	200
Incinerator residues	Container glass and metals and silica glasses	Municipal and industrial	5
Building rubble	Brick, concrete, reinforcing steel	Demolition	25

amount of transportation required, the extent of beneficiation, and the mix design requirements. For instance, many sources of mine wastes are located far from the potential markets for concrete and would entail high transportation costs. Separation of useful materials from undesirable substances (e.g., glass and metals from paper, plastics, and organic wastes in municipal wastes) would be costly unless it is done for other reasons as well. Crushed container glass tends to contain many flat elongated particles, which would lead to a harsh concrete, requiring increased amounts of cement.

Waste materials must not react adversely with other constituents of the mix. Most waste glass will readily take part in the alkali–aggregate reaction and poses a potential durability problem. The use of fly ash (itself a waste material) material as a pozzolan can provide the necessary protection. Mine tailings will contain high proportions of lead or zinc if associated with these ores, and these elements will interfere with the setting of cement. Aluminum present in incinerator residues could cause durability problems, particularly in the presence of chlorides.

Finally, the effect of waste materials on concrete properties must be considered. For example, the lower modulus of elasticity of glass compared to that of good-quality rock will lower the elastic modulus of concrete. Crushed, recycled concrete has been used as an aggregate, producing concrete with strength and stiffness equal to about two-thirds of that obtained using natural aggregates. These effects will be much more pronounced if low-strength, low-modulus materials such as rubber and plastics are used. Scrapped tires have been proposed for use in concretes where high resiliency rather than strength is required, but have resulted in low-quality concretes. Waste materials can also be used to make special lightweight or heavyweight aggregates, as will be discussed next.

The successful use of solid wastes in concrete will depend on anticipating potential problems and the ensuing properties of the concrete, and developing uses that comply with these restraints. For example, the properties of a material may preclude its

use in exterior concrete because of durability problems, but it may perform satisfactorily in concrete not exposed to weathering. There are also many applications where structural requirements are low, as in subbases or in interior partition walls. Special properties of aggregates could also be used to advantage for thermal or sound insulation, for example. However, to keep the problem in perspective, it should be remembered that use in concrete is only one possible solution to the disposal of solid wastes and that the solid-waste disposal problem is of too great a magnitude to be solved by any one approach. "Trashcrete" may have its place in the use of waste materials, but it will not dominate the field.

Lightweight and Heavyweight Aggregates

Earlier in this chapter, it was mentioned that the most common classification of aggregates was on the basis of bulk specific gravity: heavyweight, normal-weight, and lightweight aggregates. The variability in density can be used to produce concretes of widely different unit weights. The different kinds of lightweight and heavyweight aggregates are described in this section, while the properties of the corresponding concretes are discussed in Chapter 20.

Lightweight Aggregates Lightweight aggregates can be either natural or synthetic materials. The common feature of lightweight aggregates is their high internal porosity, which is the prime reason for their low bulk specific gravity. However, in some cases the solid matrix is also a low-density material. Table 7.11 summarizes some of the different kinds of lightweight aggregates that have been used in concrete. Most of these are synthetic materials, although many (those designated PN) are natural materials specifically processed to provide concrete aggregates. Expanded clay, shale, or slate is probably the most common type of lightweight aggregate used in structural concrete. The raw material is either crushed to the desired size or ground and pelletized, and it is

TABLE 7.11 Properties of Selected Lightweight Aggregates

Aggregate	*Aggregate Dry Unit Weight* (kg/m^3)	(lb/ft^3)	*Origin*[a]	*Absorption (Wt. %)*
Expanded shale, clay, slate	550–1050	35–65	PN	5–15
Foamed slag	500–1000	31–62	S	5–25
Sintered fly ash	600–1000	37–62	S	14–24
Exfoliated vermiculite	65–250	4–16	PN	20–35
Expanded perlite	65–250	4–16	PN	10–50
Pumice	—	—	N	—
Expanded glass	250–500	16–31	S	5–10
Expanded polystyrene beads	30–150	2–9	S	—
Brick rubble	~750	~45	S	19–36
Crushed stone[b]	1450–1750	90–110	N	0.5–2.0

[a]PN, processed natural material; N, natural material; S, synthetic material.
[b]Natural aggregate listed for comparison.

then heated to 1000 to 1200°C (1830 to 2190°F). At these temperatures, the material will bloat (i.e., it is puffed up to a greatly expanded size in much the same way as popped popcorn). Bloating occurs because of a rapid generation of gas (from combustion of small quantities of organic matter that often occur naturally, or which may be deliberately added) within the particle, which cannot readily escape. The material is of a composition that causes partial melting at the bloating temperature. The lowered viscosity of the material allows it to expand, and an impervious viscous coating forms on the outside, preventing the gases from escaping too quickly.

Many other synthetic lightweight aggregates are produced by similar pyroprocessing techniques. Perlite (volcanic glass), slags (calcium silicate glasses), or waste glass can be bloated. Water or air can be used as the bloating medium. In the case of perlite, hydrate water in the material is the source of bloating. Pumice, scoria, and tuffs are lavas that have been bloated naturally. Expanded vermiculite is also bloated by loss of hydrate water. Vermiculite is a clay mineral with a platey, layer structure; rapid loss of water between the layers causes the layers to be pushed apart, thereby increasing the volume by as much as 30 times.

The shape and structure of a lightweight aggregate can be quite variable and will be a consequence of the processing techniques used in its production. Aggregates may be angular or highly irregular in shape, and this will determine the workability of concrete made with the aggregate. Even more important is the porosity of the aggregate. Lightweight aggregates have high absorption values, because of the large internal interconnected porosity; this requires a modified approach to concrete proportioning. However, in some cases, entry of moisture to the body of the aggregate may be inhibited by a protective outer skin of fused material. Such an aggregate will not have a high effective absorption, although if the aggregate is fractured during mixing or handling, the effective absorption will abruptly increase as the internal pore system is exposed. Slump loss in lightweight concrete due to absorption can be an acute problem, which can be alleviated by wetting the aggregate before batching.

Lightweight aggregates are covered by ASTM C 330 (for structural concrete), C 331 (for masonry units), and C 332 (for insulating concrete). These specifications contain the requirement that aggregates should be composed "predominately of lightweight-cellular and granular inorganic material," thereby excluding materials such as wood, sawdust, cork, and the like. Organic materials can be used if they are treated to prevent possible retardation of setting and hardening, although they may swell when wet.

Heavyweight Aggregates Heavyweight aggregates are composed of materials that have a high specific gravity (see Table 7.12). These may be synthetic materials or natural materials, but synthetic aggregates are only used if concrete unit weights greater than 4000 kg/m^3 (250 lb/ft^3) are required. Choices of other materials are determined by availability and cost as well as by physical properties. Heavyweight concrete is mostly used for radiation shielding (see Sections 17.3 and 20.2); ASTM C 637 covers aggregates used for radiation shielding. It can also be used for applications where a high mass-to-volume ratio is desirable (e.g., counterweights).

TABLE 7.12 Physical Properties of Some Heavyweight Aggregates[a]

Material	*Chemical Composition*	*Classification*[b]	*Bulk Specific Gravity*	*Unit Weight* (kg/m^3)[c]
Goethite	$Fe_2O_3 \cdot H_2O$	N	3.5–3.7	2100–2250
Limonite	Impure Fe_2O_3	N	3.4–4.0	2100–2400
Barite	$BaSO_4$	N	4.0–4.6	2300–2550
Illmenite	$FeTiO_3$	N	4.3–4.8	2550–2700
Magnetite	Fe_3O_4	N	4.2–5.2	2400–3050
Hematite	Fe_2O_3	N	4.9–5.3	2900–3200
Ferrophosphorus	Fe_2O_3-P_2O_3	S	5.8–6.8	3200–4150
Steel	Fe (scrap iron steel punchings)	S	6.2–7.8	3700–4650

[a] See ASTM C 637 and C 638.
[b] N, naturally occurring; S, synthetic.
[c] $kg/m^3 \times 0.062 = lb/ft^3$.

Abrasion and Skid-Resistant Aggregates

Special aggregates are often used for toppings in heavy-duty industrial floors or other situations where a high resistance to abrasion is required. Hard, dense, strong materials such as calcined bauxite or corundum, emery, metals, flint, or quartz are used for such applications. The strength of the paste and the paste–aggregate bond are more important than aggregate hardness. The aggregate becomes an important factor in the skid resistance of concrete in older pavements, where considerable wear and polishing have occurred. It is desirable to have a mix of hard and soft minerals in the aggregate that wear and polish at different rates. A large proportion of fine aggregates in the range 3 to 10 mm (1/8 to 3/8 in.) improves skid resistance because of the tendency of fine aggregate to contain a higher percentage of harder minerals than does coarse aggregate.

Marginal Aggregates

In the past, there has been a natural tendency to avoid aggregates that *might* cause problems in concrete or require considerable beneficiation before use. As aggregates become increasingly scarce in some parts of the country, consideration is being given to rock types that have been hitherto rejected (shales, for example). When considering aggregates classed as marginal, four major areas of consideration are relevant: (1) the desired concrete properties, (2) the weaknesses of the aggregates, (3) the beneficiation of the aggregate, and (4) the use of protective measures for the concrete.

In the first place it should be remembered that some aggregates may be suitable for some concrete applications, but not for others. For example, an aggregate that contains a rather high percentage of porous material may not be suitable for concretes when strengths greater than 35 MPa (5000 lb/in.^{2}) are needed, but may be satisfactory for lower-strength concrete when there are no durability problems. Aggregates that have potential durability problems might well be satisfactory for use in concrete that will not be exposed to weathering.

At present, aggregates for concrete are required to meet ASTM C 33 or C 330, which means that they must pass various standard tests, including durability assessments. Test results should be considered in light of the anticipated service conditions and the performance of the aggregate in other regards. We have already discussed the limitations of some of these tests, and the development of improved, more precise tests might be desirable for further evaluation of marginal aggregates that fail one of the tests. For example, it has been suggested that durable shales could be identified by a series of tests involving mineralogical analysis, durability, and mechanical degradation. A thorough petrographic analysis is essential for proper evaluation of marginal aggregates. A skilled and experienced petrographer will be of great help in assessing the suitability of an aggregate for a particular application and in identifying the potential durability problems.

The discussion of beneficiation applies particularly to marginal aggregates. Density fractionation may be successful at eliminating unsatisfactory fractions, or selective quarrying may be used to advantage. The other remedy is to take protective measures when marginal aggregates must be used. The incorporation of fly ash, for example, will guard against potential alkali–aggregate attack. Crushing aggregate to a smaller maximum size may avoid freeze–thaw problems.

The use of marginal aggregates requires more care, more forethought, more ingenuity, and more cost. There is no doubt that use of marginal materials requires additional testing and closer quality control, but the extra cost involved may be less than the cost of bringing in good-quality aggregate. Necessity is the mother of invention, so as time progresses, we can look forward to the development of better tests using fundamental aggregate properties and to the development of new ways of controlling the behavior of components with poor durability.

CHAPTER APPENDIX

DETERMINATION OF MOISTURE CONTENTS ON AN OVEN-DRY BASIS.

Expressing moisture quantities on an OD basis, the absorption (A), effect absorption (EA), and surface moisture (SM) are expressed as

$$A = \frac{W_{SSD} - W_{OD}}{W_{OD}} \times 100\% \qquad \text{same as SSD basis} \tag{7A.1}$$

$$EA = \frac{W_{SSD} - W_{AD}}{W_{OD}} \times 100\% \tag{7A.2}$$

and

$$SM = \frac{W_{wet} - W_{SSD}}{W_{OD}} \times 100\% \tag{7A.3}$$

Equations (7A-2) and (7A-3) are the same as the definitions for the SSD basis, except that the denominator is now the OD weight. If the difference between SSD and

OD is small (1 to 2%), as it is for most aggregates, the actual differences between effective absorption or surface moisture calculated for SSD or OD states can be neglected.

When working on the OD basis, the quantity *total moisture* (TM) is generally used:

$$TM = \frac{W_{stock} - W_{OD}}{W_{OD}} \times 100\% \quad (7A.4)$$

Thus, it can be shown that

$$EA = A - TM \quad (7A.5)$$

when TM is measured on an air-dry aggregate, and

$$SM = TM - A \quad (7A.6)$$

when TM is measured on a wet aggregate. Or, expressing this in general terms, the moisture content of the stockpiled aggregate is given by

$$MC = TM - A \quad (7A.7)$$

When the moisture content is positive, MC represents surface moisture; if negative, Eq. (7A.7) gives the effective absorption.

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PROBLEMS

7.1. Plot the grading curves for the grading limits of the fine aggregates given in Table 7.4.

7.2. Plot the grading curves for the grading limits of coarse aggregates given in Table 7.5 for maximum aggregate sizes 1 1/2 in. and 1 in.

7.3. Two sands have the following percentages passing the fine sieves. Calculate the fineness modulus of each sand.

a. 4.75 mm (No. 4) = 97%, 2.36 mm (No. 8) = 95%, 1.18 mm (No. 16) = 92%, 600 μm (No. 30) = 85%, 300 μm (No. 50) = 30%, 150 μm (No. 100) = 5%

b. 4.75 mm (No. 4) = 95%, 2.36 mm (No. 8) = 75%, 1.18 mm (No. 16) = 45%, 600 μm (No. 30) = 20%, 300 μm (No. 50) = 10%, 150 μm (No. 100) = 3%

7.4. Two 500-g samples of different sands were found to have the following amounts retained on each sieve. Calculate the fineness modulus of each sand.

a. 4.75 mm (No. 4) = 15 g, 2.36 mm (No. 8) = 60 g, 1.18 mm (No. 16) = 100 g, 600 μm (No. 30) = 105 g, 300 μm (No. 50) = 130 g, 150 μm (No. 100) = 90 g

b. 4.75 mm (No. 4) = 5 g, 2.36 mm (No. 8) = 55 g, 1.18 mm (No. 16) = 70 g, 600 μm (No. 30) = 105 g, 300 μm (No. 50) = 200 g, 150 μm (No. 100) = 65 g

7.5. A 1000-g sample of coarse aggregate in the SSD condition weighed 633 g when immersed in water. Calculate the bulk specific gravity (SSD) of the aggregate.

7.6. A 1000-g sample from the stockpile of the same aggregate as in Problem 7.5 weighed 637 g when immersed in water. Calculate the moisture content of the aggregate in the stockpile.

7.7. A 500-g sample of sand in the SSD condition was placed in a jar, which was then filled with water. The combined weight was 1697 g. The weight of the jar filled with the water only was 1390 g. Calculate the bulk specific gravity (SSD) of the sand.
7.8. A 500-g sample of the same sand as in Problem 7.7 from the stockpile weighed 1691 g when placed in the jar and topped up with water. Calculate the moisture content of the sand in the stockpile.
7.9. The sample of coarse aggregate in Problem 7.5 weighed only 985 g after drying at 105°C overnight. Calculate the absorption capacity of the aggregate.
7.10. A 1000-g sample of coarse aggregate from the same stockpile as in Problem 7.6 weighed 992 g when dried at 105°C overnight. Calculate the effective absorption on an OD basis. (Compare your answer with the answer to Problem 7.6.)
7.11. The sample of sand in Problem 7.7 weighed 495 g after drying at 105°C overnight. Calculate the absorption capacity of the sand.
7.12. A 1000-g sample of sand from the same stockpile as in Problem 7.8 weighed 972 g when dried at 105°C overnight. Calculate the surface moisture of the sand. (Compare your answer with the answer to Problem 7.8.)
7.13. A mixture of 1080 g of gravel with an effective absorption of 0.90% and 720 g of sand with a surface moisture of 2.51% was added to a concrete mix. Compute the adjustment that must be made to the added water to maintain a constant *w/c* ratio.
7.14. Can absorption capacity be used as a measure of the total porosity of an aggregate?
7.15. What would be the consequence of using oven-dry aggregate in a concrete mix?
7.16. Why is it more desirable to batch aggregates by weight rather than by volume?
7.17. Does the fineness modulus completely describe the properties of a fine aggregate?
7.18. Why is it important to control the amount of material passing the No. 200 sieve?
7.19. An aggregate blend of 40% sand and 60% gravel has a unit weight of 1920 kg/m^3. If the bulk specific gravity of the sand is 2.60 and of the gravel is 2.70, calculate the volume percent of void space in the blend.
7.20. Suggest a strategy for controlling alkali–aggregate reactions, giving reasons for your choice.
7.21. How would you evaluate a waste material for suitability as a concrete aggregate?
7.22. A Kansas farmer suggests that you use chopped corncobs as an aggregate. Do you think that this would be a suitable material for this purpose, and what potential applications would "corncrete" have?
7.23. An Illinois tire dealer makes a suggestion similar to that of the farmer in Problem 7.22, but this time to use chopped tires. Comment.

CHAPTER 8

Chemical Admixtures

The official definition of an admixture set out in ASTM C 125 is "a material other than water, aggregates, hydraulic cement and fiber reinforcement that is used as an ingredient of concrete or mortar and is added to the batch immediately before or during its mixing." This definition encompasses a wide range of materials that are used in modern concrete technology and includes both finely divided solids (mineral admixtures) and water-soluble compounds (chemical admixtures). The former have been discussed in Chapter 5. Admixtures that are used to produce specialty concretes, such as polymer latexes and shrinkage-compensating admixtures, are discussed in the appropriate chapters. This chapter is devoted to water-soluble admixtures that are added as solutions during mixing, but a few solid compounds are included for the sake of completeness.

8.1 DEFINITIONS AND CLASSIFICATIONS

Readers may encounter two other almost identical terms: addition and additive. The term "additive" belongs to an older terminology used interchangeably with "admixture." An addition, according to ASTM C 219, is "a material that is interground or blended in limited amounts into a hydraulic cement during manufacture either as a *processing addition* to aid in manufacturing and handling the cement or as a *functional addition* to modify the properties of the finished product." A functional addition would be an admixture added by the manufacturer at the cement plant instead of by the contractor at the job site or the operator at the ready-mix plant. Thus, a Type IA portland cement has an air-entraining *addition* that should behave identically to a Type I portland cement to which an air-entraining *admixture* is added. An example of a processing addition is a grinding aid, which should not, in theory, affect the behavior of the finished cement.

There are a bewildering number of products marketed as admixtures for concrete, many of them based on proprietary formulations. However, most can be assigned to one of three broad categories of admixtures:

1. *Air-entraining agents* (ASTM C 260) are added primarily to improve the resistance of concrete to freezing and thawing.

2. *Set-controlling admixtures* (ASTM C 494) are added to control setting or to induce early hardening.
3. *Plasticizing admixtures* (ASTM C 494 and C 1017) are water-soluble polymers designed to enhance workability by increasing slump; particularly in excess of 185 mm (flowing concrete, C 1018), or to reduce the water requirements for a desired workability (C 494).

Miscellaneous admixtures include all those not included in one of these three categories and are primarily used in special applications.

8.2 USE OF ADMIXTURES

The enormous variety of chemical admixtures is due in large measure to the fact that almost every property of concrete can be modified to some extent (see Table 8.1). Thus, it is axiomatic that the reasons for using an admixture during the production of concrete will be equally varied. An admixture should be used to improve concrete construction, whether it be in the handling or consolidation of fresh concrete, in the performance of the hardened concrete, or in the cost of construction. It should be emphasized that an admixture is not a panacea for poor mix design or sloppy concrete practice, and the potential user should first ensure that the desired result cannot be attained by improvements in these areas.

Precautions in Use

All engineers who work regularly with concrete have their favorite stories to tell of admixtures preventing concrete from setting or causing some other unfortunate problem. However, the problems that do occur often arise through a lack of understanding of how an admixture interacts with concrete. To avoid unnecessary surprises, the concrete engineer would do well to take a few simple precautions when considering the use of admixtures:

TABLE 8.1 Beneficial Effects of Chemical Admixtures on Concrete Properties

Property	*Admixture Type*	*ASTM Spec*
Workability	Water-reducing admixtures	C 494
	Plasticizers	C 1017
	Air-entraining admixtures	C 260
	Pumping aids	–
	Anti-washout admixtures	–
Set Control	Set-accelerating admixtures	C 494
	Set-retarding admixtures	C 494
	Extended-set admixtures	–
Strength	Water-reducing admixtures	C 494
Durability	Air-entraining admixtures	C 260
	Water-reducing admixtures	C 494
	Corrosion inhibitors	–

1. Require admixtures to conform to relevant ASTM specifications, where applicable. A reputable manufacturer should also be prepared to supply adequate technical data for its product. This should include the following: (a) the main effect of the admixture on concrete performance; (b) any additional influences the admixture may have, whether beneficial or detrimental; (c) physical properties of the material; (d) the concentration of the active ingredient; (e) the presence of any potentially detrimental substances such as chlorides, sulfates, sulfides, nitrates, or ammonia; (f) pH; (g) potential occupational hazards for users; (h) conditions for storage and recommended shelf life; (i) preparation of admixture and procedures for introducing it into the concrete mix; and (j) recommended dosage under identified conditions, recommended maximum dosage, and effects of overdosing.

2. Follow the manufacturer's instructions regarding dosage, but make relevant tests to check that the desired effects are being obtained. It is particularly important that these tests be run using job materials under job-site conditions because the precise effect of an admixture will depend on many factors: cement composition, aggregate characteristics and impurities, mix proportions, presence of other admixtures, type and length of mixing, time of addition, concrete temperature, and environmental conditions.

3. Ensure that reliable procedures are established for accurate batching of the admixture. This is particularly important for air-entraining admixtures and chemical admixtures, where typical dosages may be well below 0.1% by weight of cement. In such cases, overdoses can easily occur and the effects may be disastrous.

4. The effects that the admixture may have on other concrete properties should be taken into account, particularly if these effects may be unfavorable. Some commercial admixture formulations contain materials that belong to different categories, and their effects may change significantly at different temperatures.

Specifications and Analysis

Although many new compounds are being developed for use in concrete, admixtures often are derived from waste products from other industries, and as such may be complex mixtures of compounds. Suppliers blend them with other chemicals to mitigate undesirable side effects and to ensure uniform quality. Thus, most admixture formulations are multicomponent mixtures. Admixture specifications are therefore performance specifications, which set minimum changes in the property of interest (e.g., decrease in setting time) and limits on how other properties should change (e.g., strength). For example, air-entraining admixtures should conform to ASTM C 260, which is essentially a performance specification in which the admixture is tested according to the provisions of ASTM C 233. The key test in this method is the performance of the concrete exposed to rapid freezing and thawing (ASTM C 666). Admixtures are compared to neutralized vinsol resin, which has been selected as the reference air-entraining admixture. ASTM C 226 is the relevant specification for air-entraining additions.

8.3 AIR-ENTRAINING ADMIXTURES

One of the major disadvantages of concrete is its susceptibility to damage during freezing and thawing cycles when it is in a saturated or near-saturated condition. Concrete can be badly damaged after a single winter's exposure if corrective measures are not taken, and this rapid deterioration would effectively preclude the use of concrete in most major applications: pavements, dams, foundations, and so on. All porous materials (brick, ceramic tile, etc.) suffer from the same deficiency unless the material is specially designed to resist freezing and thawing (frost resistance). Fortunately, concrete can be made frost resistant by the addition of air-entraining admixtures (see Figure 8.1). All concrete designed for outside exposure should be air-entrained routinely in the northern United States and throughout Canada. The mechanism of frost protection is described in detail in Chapter 18; here, we are only concerned with describing the effect air-entraining agents have on concrete properties.

Air-entraining admixtures increase the amount of air that is incorporated into concrete during mixing. The volume of air required to give optimum frost resistance has been found to be about 9% by volume of the mortar fraction (Table 8.2), although the air is actually entrained within the paste fraction. For routine field tests, the easiest quantity to measure is the amount of air expressed as a percentage of the *concrete* volume. The air content must be in the range 2 to 8% by volume of concrete for satisfactory frost protection, the actual amount depending on the maximum size of the coarse aggregate. This is because more paste is required to provide workable concrete with a lower maximum size of coarse aggregate. The air-content measurement includes voids naturally entrapped within the paste during mixing. Intentionally entrained air increases the total volume of air voids by 3 to 4% of the volume of the concrete and is distributed in the paste as very fine bubbles.

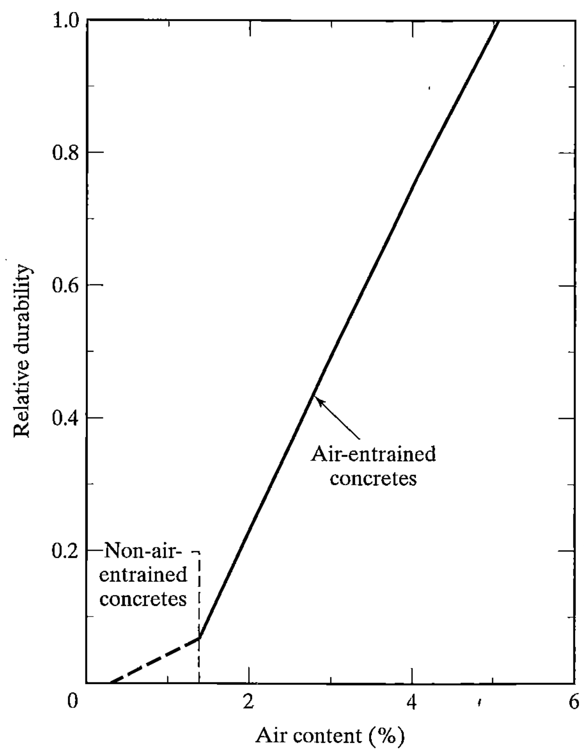


FIGURE 8.1

Effect of air entrainment on the frost resistance of concrete.

TABLE 8.2 Characteristics of Air-Entrained Concrete at Optimum Frost Resistance (Cement Content 307 kg/m^3 or 517 lb/yd^3)[a]

		Measured Air Contents (% by volume)				
Maximum Aggregate Size, mm (in.)	*Air Contents (ACI) by Volume ±1%*[b]	*Non-Air-entrained Concrete*	*Air-Entrained Concrete*	*Air-Entrained Mortar*	*Air-Entrained Paste*	*Bubble Spacing Factor, mm (in.)*
63.5 (2½)	4.0	0.5	4.5	9.1	16.7	0.18 (0.007)
38 (1½)	5.0	1.0	4.5	8.5	16.4	0.20 (0.008)
19 (¾)	6.0	2.0	5.0	8.3	16.9	0.23 (0.009)
9.5 (⅜)	7.5	3.0	6.5	8.7	19.7	0.28 (0.011)
Mortar	–	–	–	9.0	23.0	0.30 (0.012)

[a]Based on data from P. Kleiger, in *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, ASTM STP 169C, American Society for Testing and Materials, West Conshohocken, PA, pp 484-490 (1994).
[b]Values recommended by ACI for air-entrained concrete

The Entrained Air-Void System

The structure of the air-void system is critical for the satisfactory performance of concrete under freezing and thawing conditions. Air-entrained concrete cannot be distinguished from non-air-entrained concrete with the naked eye, because of the fineness of the bubble system. When viewed under a microscope (Figure 8.2), tiny bubbles can be seen randomly dispersed throughout the paste. They are approximately spherical in shape and range in diameter from 0.05 mm to 1.25 mm (0.002 to 0.05 in.). It looks as if the paste has been blended with a fine foam and this essentially is what happens. The air-entraining agent causes the water to foam, stabilizing the bubbles so that they are locked into the paste during hardening. A good air-void system will provide literally millions of bubbles in a cubic meter of concrete.

The critical parameters defining a satisfactory air void system are the spacing factor, specific surface area, and bubble frequency, which are determined microscopically in accordance with ASTM C 457. The spacing factor ($\bar{L}$), which is defined as the average distance from any point in the paste to the edge of the nearest void, should not exceed 0.2 mm (0.008 in.) to ensure adequate freeze–thaw protection (Figure 8.3).[1] The specific surface area (α) and bubble frequency (n) are interrelated and reflect the mean size of the bubbles. In a protective air void system, α should not be less than 25 mm^2/mm^3 (600 in^2/in.^{3}) and n should be 0.3–0.6 mm^{-1} (8–16 in.$^{-1}$). Although the

[1]Experience has shown that this limit can be relaxed somewhat in superplasticized concrete and at low *w/c* ratios.

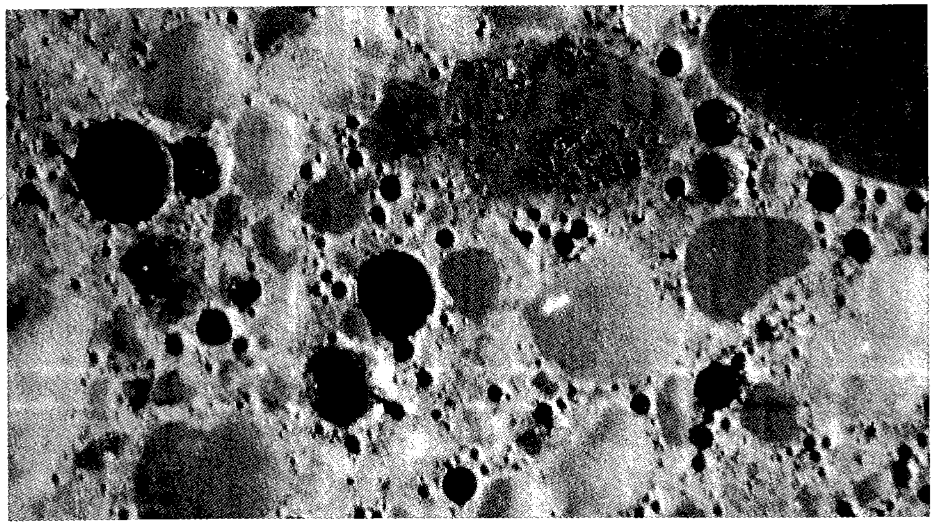


FIGURE 8.2

Micrograph of air-entrained concrete. (Photograph courtesy of Portland Cement Association.)

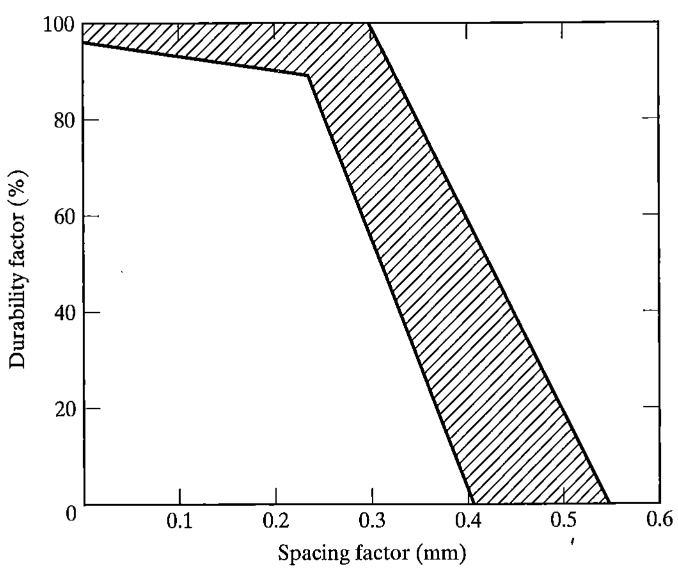


FIGURE 8.3

Relationship between durability and bubble spacing factor of entrained air.

spacing factor is the most commonly used parameter (see Figure 8.3 and Table 8.2), all three target values should be achieved for a properly air-entrained concrete. This ensures a well-distributed system of small bubbles that protects the whole paste volume. These values are attained when the recommended air contents in Table 8.2 are maintained using an air-entraining agent from a reputable supplier. Quantitative microscopic examination (ASTM C 457) can be carried out routinely to determine these parameters, using automated computer-based systems, although in the field only the total air content is controlled. Some concretes (e.g., those containing superplasticizer) may show good frost resistance even when the air void parameter limits are exceeded. Alternative parameters that give an estimate of the fraction of paste protected have been proposed to account for the random distribution of air bubbles in the paste, but none have yet been adopted by ASTM.

Air-Entraining Materials

It follows from the preceding discussion that air-entraining admixtures must contain compounds that will promote the formation of stable foams. Bubble formation in water is normally a transient phenomenon because the high surface tension of water opposes the creation of the air–water interface that bounds a bubble. Foams can only be formed if the energy barrier represented by surface tension can be overcome so that masses of stable bubbles can be created during agitation. Air-entraining agents contain *surface-active agents* that concentrate at the air–water interface, lower the surface tension so that bubbles can form more readily, and stabilize the bubbles once they are formed. Surface-active agents are molecules which at one end have chemical groups that tend to dissolve in water [*hydrophilic* (water-loving)], while the rest of the molecule is repelled by water [*hydrophobic* (water-hating)] (Figure 8.4a). The molecules tend to align at the air–water interface with their hydrophilic groups in the water and the hydrophobic portion in air (Figure 8.4b).

Carboxylic acid or sulfonic acid groups are commonly responsible for the hydrophilicity of the molecules and are attached to hydrophobic aliphatic or aromatic hydrocarbons. Air-entraining admixtures are thus closely related to synthetic detergents, although the foaming capacity of the latter is only a side effect of other "surface-active" properties. Most household detergents cannot be used as air-entraining admixtures, however, because the foams are too coarse (large bubble size) and not sufficiently stable to remain intact while the concrete is hardening.

Since many commercial air-entraining admixtures are prepared from by-products of other industries (e.g., pulp and paper, petroleum, and animal processing), the active ingredient may only be a fraction of the total admixture. However, because the admixture acts only at surfaces, a little goes a long way. Since a typical dose is only 0.005 to 0.05% of active ingredient by mass of cement, admixtures are often prediluted to assist accurate batching and are added to the concrete with the mix water. Some manufacturers intergrind an air-entraining agent with the cement to obtain air entrainment directly on mixing with water without the need of an admixture. Such cements are designated IA, IIIA, etc., but they may not always entrain sufficient air and should not be used without being tested under job conditions. Because of this limitation, the use of air-entrained cements is usually limited to small jobs.

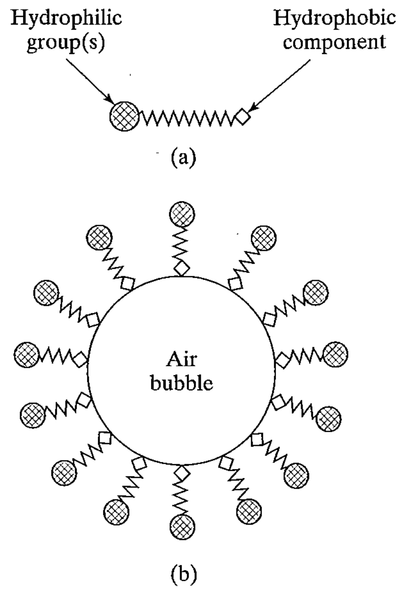


FIGURE 8.4

Schematic representation of air entrainment by surface-active molecules: (a) characteristics of a surface-active molecule; (b) stabilized air bubble.

Factors Affecting Air Entrainment

An air-entraining admixture should always be tested under field conditions because the amount of entrained air it provides can be affected by a variety of factors (see Figure 8.5). The admixture dose can be adjusted to bring the air content to within the recommended limits. Increasing the admixture dose will invariably increase air content, decrease bubble size, and decrease the spacing factor. However, the variety of surface-active agents present in commercial admixtures means that job conditions may have different effects on different admixtures.

Materials The air-entraining potential of an admixture depends on the concrete materials used and their proportions. Finely ground cements entrain less air than do those ground more coarsely. Similarly, a high proportion of fines in the fine aggregate or the addition of finely divided mineral admixtures (such as fly ash) will reduce air contents. More air is entrained in concretes with a high sand content (Figure 8.5c), but the gradation (material between the 600 μm (No. 30) and 300 μm (No. 50) sieves is the most effective) and particle shape of the aggregates will also affect air. Impurities in the mixing water or in the aggregates may have positive or negative effects on air entrainment.

The use of other admixtures simultaneously with air-entraining agents should be approached with caution, because they may affect air-entraining abilities. When water-reducing agents are used, the dosage of air-entraining agent frequently can be reduced while maintaining a given air content. Some common chemical admixtures (e.g., superplasticizers) may interact when mixed with air-entraining admixtures and inhibit entrainment. In such cases, the second admixture should be added to the concrete after the air has been entrained. Carbon present in fly ashes will suppress air entrainment.

Cement content has a marked effect on the total air entrained. At a given dose, lean concretes entrain more air than do rich mixes, and low *w/c* ratio concretes entrain

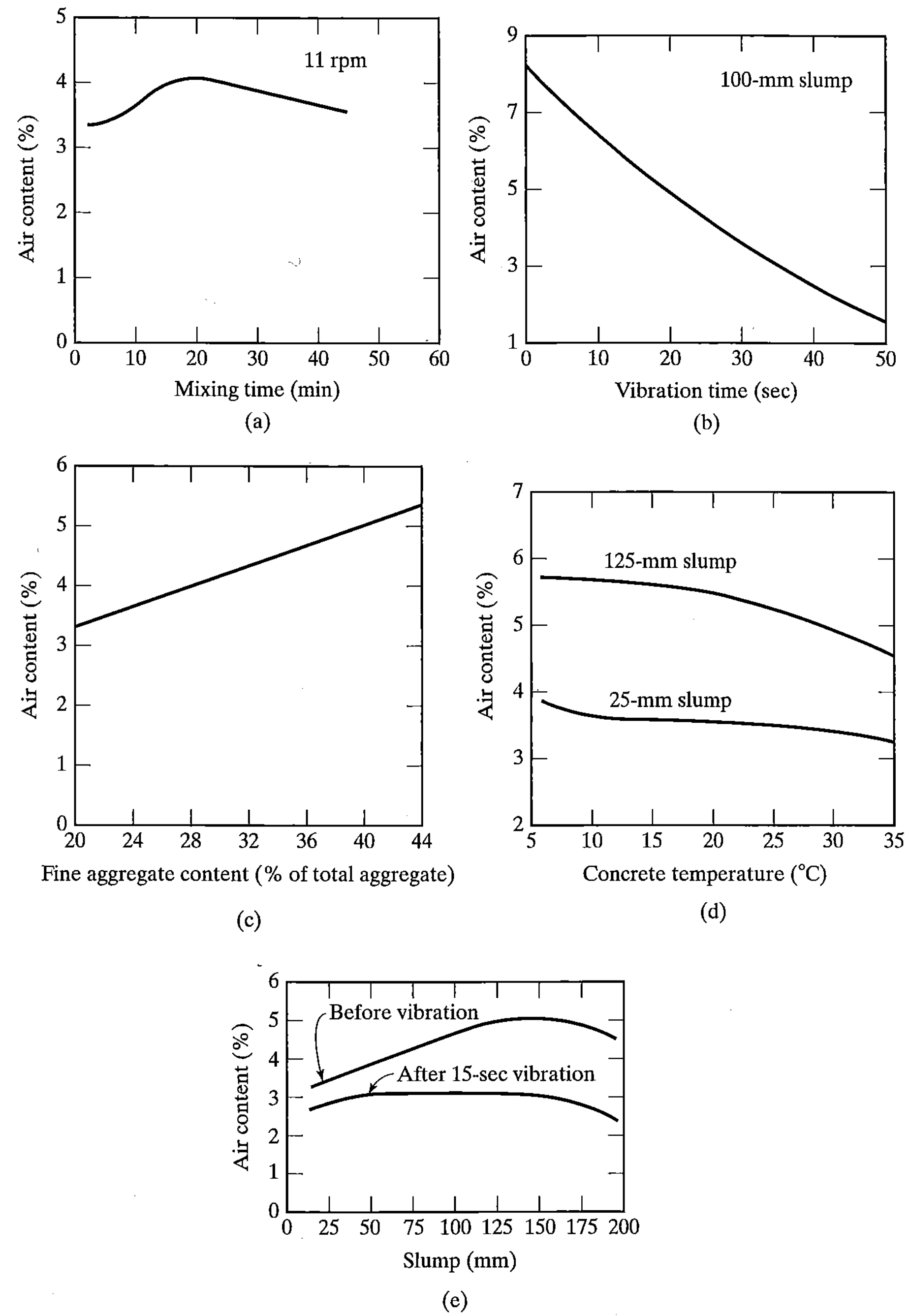


FIGURE 8.5

Effect of concrete parameters on the volume of entrained air: (a) mixing time; (b) time of vibration; (c) fine aggregate content; (d) concrete temperature; and (e) concrete slump.

less air than do concretes with high *w/c* ratios. (The latter frequently have higher sand contents.) However, such differences in total air content do not necessarily imply that the spacing factor has changed, because the void size may change.

Mixing and Consolidation Air entrainment occurs while the concrete is being agitated during mixing. The total volume of entrained air depends on the type of mixer, the rate of mixing, and the amount of concrete being mixed. The time of mixing is also important: The air content will initially increase with time and then gradually decrease during prolonged mixing (Figure 8.5a). Maximum air entrainment is generally achieved during normal mixing times, and if air loss should occur during extended agitation (e.g., during transit of ready-mixed concrete), this can readily be allowed for by adjusting the admixture dose.

Increasing the initial slump of the concrete will result in a greater air content over normal slump ranges (25 to 150 mm) (see Figure 8.5e). Temperature is also an important parameter that affects air entrainment; air content varies inversely with temperature (Figure 8.5d). Increasing the temperature from 10 to 38°C (50 to 100°F) will approximately halve the air content. The effect of temperature depends on slump. Vibration also affects the final air content (Figure 8.5b,e). However, coarse bubbles are selectively eliminated, and only extended vibration will remove a sufficient number of fine bubbles to compromise frost resistance.

Effect on Concrete Properties

Although air-entraining admixtures are added primarily to improve the durability of concrete, they have other important influences on the behavior of both fresh and hardened concrete.

Fresh Concrete Air entrainment improves the workability and cohesiveness of fresh concrete, at both high and low slumps. Air-entrained concrete is more workable than a non-air-entrained mix at the same slump and can be placed and compacted more readily. Bleeding and segregation are considerably reduced in wet mixes during handling and transportation. Addition of 3–4% entrained air (total air content 4–7%) will increase the slump of concrete about 35 to 50 mm ($1\frac{1}{2}$ to 2 in.) at a given paste content, the exact amount depending on the mix design and materials. This is due partly to the fact that the surface-active ingredient to some extent acts at the solid–water interface in the same way as a water-reducing admixture (see Section 8.4). But the main reason for improved workability is that the tiny bubbles behave as low-friction, elastic, fine aggregate that reduces interactions between conventional solid aggregate particles during placement operations. Thus, air-entrained mixes behave as if the mix were oversanded, and its beneficial effects are most apparent in lean or harsh mixes deficient in fines.

Rather than using air-entraining agents to increase slump, it is more common to reduce the amount of sand and water required to produce a desired slump. Water contents can generally be reduced 12 to 25 kg/m^3 (20 to 40 lb/yd^3), for a 4% increase in air content. This partially offsets the strength reduction that accompanies air entrainment (see the discussion that follows). The improvements in workability are such that air entrainment may be used even when freeze–thaw protection is not required. This is particularly

beneficial where mixes have low cement contents, poor aggregate gradings, or harsh aggregate textures. Air entrainment will help produce a more uniform, well-compacted concrete.

Hardened Concrete The introduction of additional void space with air entrainment must have a detrimental effect on strength. The increased air voids are partially offset by the decrease in capillary porosity from the lower *w/c* ratio that can be used. Generally, for a fixed *w/c* ratio, a strength loss of 10 to 20% can be anticipated for most air-entrained concretes. A good "rule of thumb" is that each 1% of entrained air will lower strength by 5%. However, for a fixed air content and slump, air-entrainment has much less effect and may not affect the strength of lean concrete mixes (cement contents below 300 kg/m^3 or 500 lb/yd^3). In such cases, the higher air contents needed for good frost protection may allow expected strength decreases to be entirely offset by decreases in *w/c*. There is an optimum amount of entrained air for good durability of concrete (Figure 8.6). Excessive amounts of air will drastically lower the strength of concrete and reduce its resistance to stress from freezing. The high air contents generally indicate larger bubble sizes rather than smaller spacing factors. As well as improving freeze–thaw durability, air entrainment helps combat the scaling caused by deicing salts used on pavements and bridge decks. It also improves the sulfate resistance of concrete. The lower *w/c* ratios that can be used and the better compaction characteristics result in more impermeable concrete and a better overall resistance to aggressive agents.

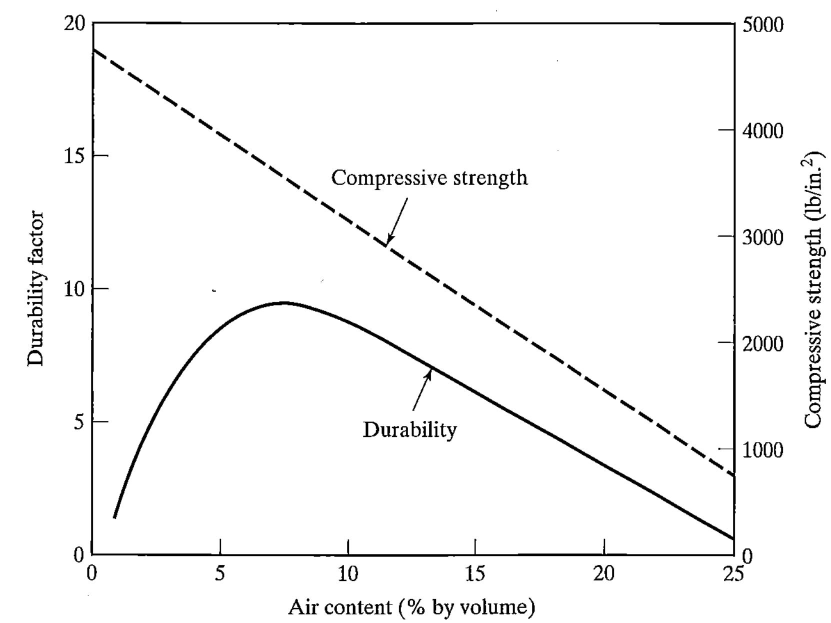


FIGURE 8.6

Effect of entrained air on durability and strength (*w/c* ratio, slump, and sand percentage held constant). (Adapted from Concrete Manual, 8th ed. U.S. Bureau of Reclamation, Denver, CO, 1975.)

TABLE 8.3 Effect of *w/c* Ratio on the Air Void System in Concrete

w/c Ratio	*Air Content (%)*	*Spacing Factor mm (in.)*	*Linear Expansion per Freeze–Thaw Cycle (%)*
0.35	4.8	0.11 (0.0043)	0.00004
0.45	4.7	0.14 (0.0055)	0.00014
0.55	5.2	0.15 (0.0059)	0.00021
0.65	4.9	0.18 (0.0071)	0.00026
0.75	5.3	0.23 (0.0091)	0.00036

Recommended Air Contents

Closely spaced air voids protect concrete during freezing by providing reservoirs for water that exits from the paste on freezing. This mechanism will be discussed in detail in Chapter 18 and is the reason for the importance of the spacing factor. Yet, in practice, we have to rely on total air contents, such as are used in the ACI standard practice for mix design (Table 8.2), which depend on many factors and which need not be related directly to the spacing factor. An example of this important point is illustrated in Table 8.3. It can be seen that for a given air content, increasing the *w/c* ratio increases the spacing factor (and increases bubble size), thus lowering the resistance to freezing and thawing. It can be seen, as a corollary, that the air content must be increased in concretes with high *w/c* ratios to maintain a given spacing factor. The recommendations in Table 8.2 should therefore be used sensibly. They should be applied to concretes that will be exposed to severe conditions where freeze–thaw cycles may be frequent, deicers will commonly be used or sulfate attack is anticipated. For low-slump concretes (less than 25 mm slump) with low *w/c* ratios and high cement contents, air contents can be safely reduced below the recommended amounts. A suitable air void system can generally be achieved in such concretes by using the same dose that would be required to attain the recommended air content in Table 8.2 at a higher slump (e.g., 85–100 mm or 3-4 in.), even though the air content in the low-slump concrete will fall below recommendations. If air entrainment is used only to improve workability and handling, recommended air contents can be reduced by about one-third.

8.4 WATER-REDUCING ADMIXTURES

As the name implies, a water-reducing admixture lowers the water required to attain a given slump; that is, it reduces the water demand of the concrete. This property can be used to advantage in several ways. Achieving the desired slump with less water at a constant cement content means an effective lowering of the *w/c* ratio, with a consequent general improvement in strength, impermeability (water-tightness), and durability. Alternatively, the desired slump may be achieved without changing the *w/c* ratio by lowering the cement content. This may be done for economic reasons (by cutting back on the most expensive ingredient of concrete) or for technical reasons (e.g., to lower the heat of hydration of the concrete). Finally, a water-reducing admixture may be used to increase slump without increasing cement and water contents, to facilitate difficult placements. (When used this way, it is sometimes called a plasticizer.)

TABLE 8.4 Typical Effects of Water-Reducing Admixtures

Classification	*Common Name*	*Typical Dose* (%)*	*Slump Increase (mm)*	*Water Reduction percent*	*Water Reduction w/c*	*ASTM Spec.*
Low-range	Regular	0.1%	50–85	5–10	−0.05	C 494
Mid-range	Mid-range	0.5%	50–100	10–15	−0.10	
High-range	Superplasticizer	1.0%	>100	15–30	−0.15	C 494, C 1017

*Active ingredient by weight of cement.

Classification of Water-Reducers

Three types of water-reducing admixtures are marketed by several companies and the capabilities of each type are summarized in Table 8.4. The improved performance of the mid-range and high-range products is the ability to add sufficient quantities to increase the slump without adversely affecting setting times or air entrainment.

Low-Range (Regular) Admixtures These water-reducing admixtures (or water reducers) are classified as Type A in ASTM C 494. Retarding (Type D) and accelerating (Type E) versions are also produced. ASTM C 494 is a performance specification that sets a minimum for the decrease in the water content required to achieve the same slump as a control mix while maintaining other properties within specified limits. When used to increase workability at a given water content, they are acting as *plasticizers*. No limits are set for an increase in slump at the same water content.

According to ASTM C 494, an admixture can be classified as water-reducing if it reduces water requirements by 5%; under this criterion, many air-entraining admixtures could also be classed as water-reducing admixtures. Regular water-reducing admixtures achieve reductions of 5 to 10% in water requirements at normal dosages. The actual water reduction achieved at any dosage depends on the type of admixture, cement fineness, mix proportions, temperature of the concrete, and time of addition. Even the composition of the cement can be important; it has been found that water reduction is lower for cements high in C_3A or in alkalis. During the early hydration of C_3A, the admixture can be removed from the cement surface by being absorbed in the hydration products.

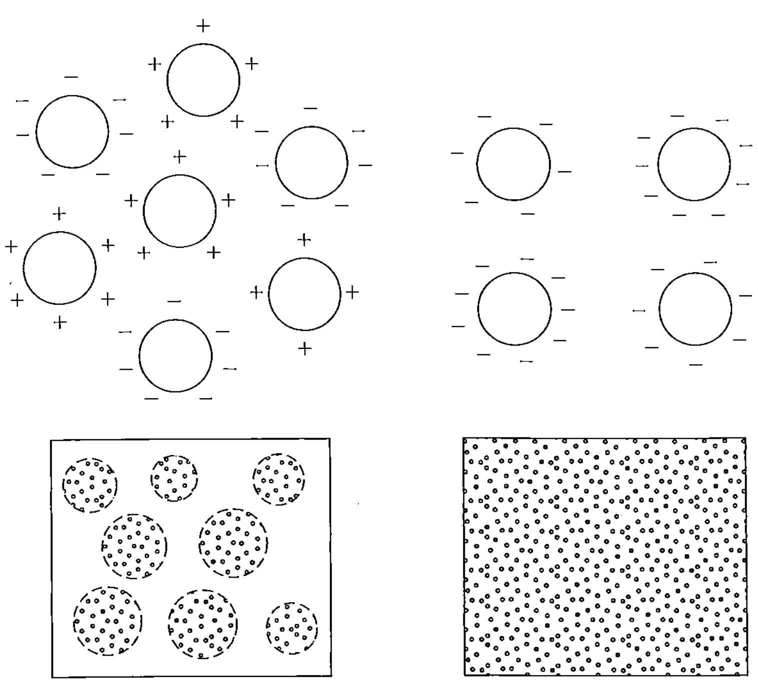
Mid-Range Water Reducers Admixture companies now offer formulations that function best in the slump range 125 to 200 mm (5 to 8 in.), that is, between conventional concrete and flowing concrete. These are called mid-range water-reducing admixtures, although this classification is not recognized by ASTM C 494.

Superplasticizers (High-Range Water Reducers) Admixtures that can achieve much higher amounts of water reduction (12–30%), are called high-range water-reducing agents in ASTM C 494 [Type F and Type G (retarding)]. These admixtures are used frequently to produce high-strength concrete, since workable mixes with *w/c* ratios well below 0.40 are possible. These same admixtures can be used without water reduction to produce concretes with very high slumps: in the range 150 to 250 mm (6 to 10 in.). At these high slumps, concrete flows like a liquid and will fill forms efficiently, requiring

very little vibration. Highly workable mixtures are called *flowing concretes* and require slumps to be in excess of 190 mm (8.5 in.). This is the reason for the common name *superplasticizer*, which is universally used outside formal documents and will be used in this text also. When used for this purpose, superplasticizers must conform to ASTM C 1017, which sets a performance limit in terms of an increase in slump (> 90 mm or 3.5 in.). There are no fundamental differences between admixtures conforming to ASTM C 1017 and C 494 specifications, and most commercial products will meet both requirements. The reason for two specifications is the convenience of specifying specific performance criteria in terms of the plasticizing effects.

Mechanism of Water Reduction

Water-reducing admixtures are negatively charged organic molecules that adsorb primarily at the solid–water interface. Solid particles carry residual charges on their surfaces, which may be positive, negative, or both. In cement paste, opposing charges on adjacent particles of cement can exert considerable electrostatic attractions, causing the particles to flocculate (Figure 8.7a). A considerable amount of water is tied up in these agglomerates and adsorbed on the solid surfaces, leaving less water available to reduce the viscosity of the paste and hence that of the concrete. Molecules of the water-reducing admixtures interact to neutralize these surface charges and cause all



(a) (b)

FIGURE 8.7

Dispersing action of water-reducing admixtures (schematic representation): (a) flocculated paste; (b) dispersed paste.

surfaces to carry uniform charges of like sign. Particles now repel each other, rather than attract, and remain fully dispersed in the paste (Figure 8.7b). Thus, most of the water is available to reduce the viscosity of the paste and of the concrete. These surface interactions are most noticeable with the fine cement particles and mineral admixtures, since the tendency for flocculation is common to all finely divided solids. For example, deflocculating agents or plasticizers are commonly used with clay–water systems to produce free-flowing slurries at low water contents. The use in concrete technology is analogous and in some cases involves the same chemicals. Some of the newer admixtures use negatively charged polymers. In this case, the bulky nature of the adsorbed molecules can cause additional *steric repulsion*, which enhances the repulsion between particles.

Composition

Regular water-reducers can be divided into three types, based on the general composition of their active ingredients. These are (1) lignosulfonates; (2) hydroxy-carboxylic acids and their salts; and (3) hydroxylated polymers derived from hydrolyzed starch (polysaccharides). Most superplasticizers are low molecular-weight synthetic polymers containing oxyacid groups attached to the carbon backbone at regular intervals. Most commercial admixtures belong to one of four polymer types: (a) sulfonated naphthalene-formaldehyde condensates, (b) sulfonated melamine-formaldehyde condensates, (c) lignosulfonates (purified to remove undesirable side effects), and (d) carboxylated acrylic ester copolymers (polycarboxylates). The acid groups are responsible for producing a negative surface charge on the cement particles and thereby dispersing the particles by *electrostatic repulsion*, as is the case with conventional water reducers. Superplasticizers, being bulky molecules, provide steric repulsion, as well, to achieve complete dispersion, and this may be the major mechanism in the case of the polycarboxylates. (These latter compounds have long graft side chains attached to the carbon backbone, giving the molecule a "comb-like" appearance.) Dispersion of the cement grains occurs with the side effects of set-retardation and excessive air entrainment usually being less troublesome than for conventional plasticizers.

Effects on the Properties of Fresh Concrete

Flowing Concrete Concretes with very high slumps can be used to advantage in difficult placements (e.g., sections with very congested reinforcing steel), in placements where adequate vibration cannot be achieved (e.g., underwater placements), or to improve the ease of placement (e.g., pumping). Slumps exceeding 150 mm (6 in.) can be achieved with conventional water reducers only by using oversanded mixes with very high cement contents. The use of superplasticizers allows higher slumps to be achieved using mix proportions typical of normal slump concretes without the occurrence of excessive segregation and bleeding. Such flowing concretes are particularly well suited for tremie placements, rapid pumping of concrete, and "self-leveling" of flatwork. A reasonable starting point is a conventional mix with a slump of 85 mm ($3\frac{1}{2}$ in.) that is designed for pumping (i.e., has a slightly higher sand content). The admixture is added after initial mixing to bring the slump up to the desired level. (This is in contrast to the use of other chemical admixtures that are added with the mix water.) If concrete is

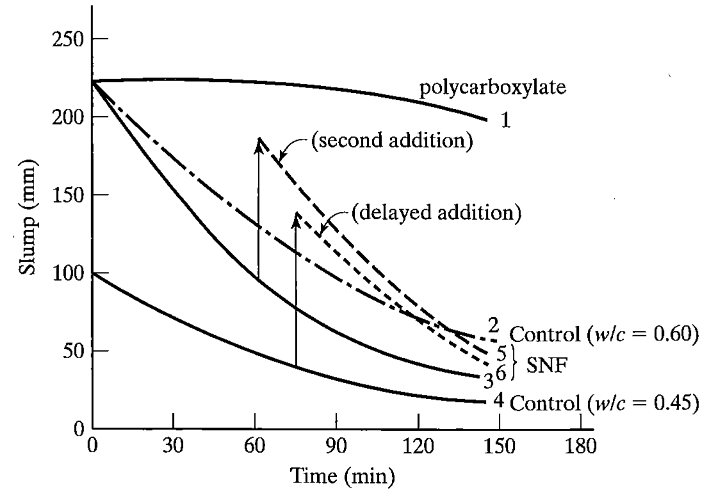


FIGURE 8.8

Schematic view of effects of superplasticizer. All curves except (4) are concretes with slump = 225 mm (9 in.). Dosages adjusted to give same high slump for mixes (1), (2), and (3). High slump control mix (2) requires w/c = 0.60. Rapid slump loss for the sulfonated naphthalene-formaldehyde condensate (SNF) admixture (3) can be compensated for by second addition (5) or delayed addition to low slump concrete (6).

overdosed with a superplasticizer, it may result in a mix that is so fluid that it segregates. Fortunately, this problem can be corrected by continuing mixing until the concrete stiffens enough to regain its cohesive properties.

Slump Loss A rapid rate of slump loss can be a major problem with superplasticized flowing concrete (see Figure 8.8). While slump loss is a problem with any high-slump concrete, it may be aggravated by the use of some superplasticizers. This problem can be overcome by adding a second dose if necessary at the job site or by delaying the initial addition of the admixture until just before placement. Repeat additions are possible without comprising concrete properties (Figure 8.8), provided that the admixture is properly blended into the concrete in the truck mixer. Incremental additions can also be used. Retarding formulations (ASTM C 494 Type G) reduce slump loss, but the newer superplasticizers are designed to maintain a high slump for extended periods, making addition at the ready-mixed plant more effective. These strategies are illustrated in Figure 8.8.

Sometimes admixtures that also have retarding tendencies can cause workability problems, due to acceleration of the early hydration of C_3A. In extreme cases, flash set can occur. Delaying the addition of the admixture will often ameliorate this problem, or an alternate admixture can be used. Retempering concrete with water to restore workability is an undesirable practice because of the resulting increase in the w/c ratio. Using additional water-reducing agents during retempering can reduce or eliminate the need to add further water, provided that excessive set retardation does not occur.

Bleeding and Air Entrainment While water-reducing admixtures improve the workability of concrete as measured by slump, they may not necessarily improve cohesiveness. Admixtures based on hydroxycarboxylic acids tend to increase bleeding and should be used with care in high-slump concretes. Although well-designed flowing concrete made with superplasticizers will not segregate, overdosing must be avoided since it will cause excessive bleeding and segregation. Lignosulfonate-based admixtures tend

to bleed less because they entrain air. Any regular water-reducing admixture, regardless of its own air-entraining ability, will reduce the amount of an air-entraining admixture required to attain a given air content if it is added at the same time. This is because the air-entraining admixture is no longer adsorbed on the solid surfaces of the cement and is therefore wholly available to act at the air–water interface. On the other hand, superplasticizers have air-detraining properties and an air-entraining agent must be added to the concrete to get a stable air void system before a superplasticizer is added.

Retardation of Set Many of the regular (and some mid-range) water-reducing admixtures will also act as retarding admixtures. If prolonged setting times are not convenient, the admixture can be formulated with an accelerating admixture to counteract the retarding tendencies or even to provide some net acceleration of setting. However, non-retarding formulations sometimes show a tendency to cause retardation at low curing temperatures. It has been found that delaying the addition of the admixture until a few minutes after water has been first added will increase potential water reduction, enhance air entrainment, and increase the retardation of set (if this occurs on regular additions).

Effect on the Properties of Hardened Concrete

Compressive Strength When admixtures are used to lower water requirements, increases in compressive strength can be anticipated. These increases can be observed in as early as one day if excessive retardation does not occur. It is generally agreed that increases in compressive strength are up to 25% greater than would be anticipated from the decrease in w/c ratio alone. Probably, this reflects the development of a more uniform microstructure when the cement is dispersed. Some refer to this phenomenon as increasing the efficiency of the cement. Thus, when used at low w/c ratios, superplasticizers can modify Type I cement to behave as super-high early-strength cement exceeding even the strength gain of a Type III. It should be emphasized that this is the result of both a greater cement efficiency and a greatly reduced void content; the rate of hydration need not be increased. Unlike Type III cement, the more rapid strength gain will not be accompanied by an increase in the rate of evolution of heat, if low cement contents are used.

High-Strength Concretes. Very high strengths can be achieved when w/c ratios are lowered below 0.40. To do this economically and without excessive generation of heat requires the cement content to be kept to normal levels, while providing an acceptable slump. Superplasticizers are well suited to achieve these objectives and are used in most concretes with 28-day compressive strengths exceeding 80 MPa (11,000 psi). Low w/c ratios also improve the rate of strength gain at early ages and can be very useful in the precast concrete plant. After 24 hours of normal curing, it is possible to obtain compressive strengths that take 8 days to develop using a higher w/c ratio (e.g., 0.50). Even larger gains are realized if steam curing is used also.

Other Properties It is often stated that water-reducing admixtures increase the drying shrinkage and creep of concrete. However, the data are conflicting and depend

very much on the cement type and the particular admixture. An increase in the *rate* of shrinkage is often observed; after 90 days of drying, there is little difference in shrinkage compared to a control concrete. With creep, the situation is less clear and different admixtures may increase or decrease creep. If drying shrinkage or creep values are critical, then tests should be carried out to assess the effects and design the concrete to reduce the values if necessary. The reduction of the *w/c* ratio and the creation of a more uniform pore structure means that the permeability of concrete can be reduced by the use of water-reducing admixtures, with a general improvement of durability. Concretes must, of course, be air entrained for frost protection, but the spacing factor required for good frost resistance is found to be higher than the value required for conventional concrete.

8.5 ADMIXTURES FOR SET CONTROL

Set-Retarding Admixtures

Retarders can be used whenever it is desirable to offset the effects of high temperatures that decrease setting times or to avoid complications when unavoidable delays may occur between mixing and placing. Prolonging the plasticity of fresh concrete can be used to advantage in placing mass concrete. Successive lifts can be blended together by vibration, with the elimination of cold joints that would occur if the first lift were to harden before the next were placed. Retarders can also be used to resist cracking due to form deflection that can occur when horizontal slabs are placed in sections. Concrete that has set, but has acquired little strength, is liable to crack when subsequent placements cause the formwork to deflect. If the plastic period is prolonged, the concrete can adjust to form deflections without cracking.

Composition Set-retarding admixtures (Type B in ASTM C 494) can be classified according to their chemical constitution. The major categories are (1) lignosulfonic acids and their salts; (2) hydroxycarboxylic acids and their salts; (3) sugars and their derivatives; (4) phosphates and organic phosphonate salts; and (5) salts of amphoteric metals, such as zinc, lead, and tin. Chemicals falling into categories 1–3 also possess water-reducing capabilities and can be classified as water-reducing, set-retarding admixtures (Type D in ASTM C 494).

Mode of Action

Effect on C_3S Hydration. Retarders slow down the rate of early hydration of C_3S by extending the length of the induction period (Stage 2). Thus, the setting time, as measured by penetration tests or by calorimetry, is extended, as is early strength development. Organic retarders are able to strongly adsorb onto the nuclei of calcium hydroxide and inhibit their growth into large crystals. Further hydration is thus prevented until this effect is overcome; the length of Stage 2 therefore depends on the amount of retarder added. Once Stage 3 begins, hydration can proceed normally again. When inorganic retarders are used, the situation can be more complex, since they also can form coatings around the C_3S particles that severely reduce the rate of reaction.

Extended Set Control. Overdosing concrete with a retarding admixture can prevent set entirely. Deliberate overdosing has helped many a ready-mix truck driver out of a tight spot. A bag of table sugar added to the truck can take care of those emergencies when excessive delays may cause the concrete to set up in the truck. Carbonated beverages will be even more effective since they contain phosphoric or citric acids as well as sugars, all of which are retarders.

This strategy can be implemented in a more controlled way using an extended set-control admixture, which is a two-component formulation designed to initiate set at a specified time. First, a large dose of a retarding admixture (the stabilizer) is added, which will delay the setting of concrete for hours or days. At the appropriate time, a set accelerator (the activator) is added to overcome the retardation and initiate set within a few hours. Although in practice an overdose of any retarder should work, combinations of carboxylic acids and organic phosphonates are used to avoid complications and ensure predictable performance. While they can be used to protect concrete during long hauls or construction delays, these admixtures are primarily used by the ready-mixed industry to hold unused concrete and eliminate the need to wash out the trucks at the end of each day. The retarding component is added to the mixer trucks at the end of the day, and the next morning the accelerating component is added to the first batch of concrete used to refill the trucks. Adding more cement or a new concrete batch without the accelerator to the old batch can also reinstate hydration.

Effect on C_3A Hydration. Retarders also tend to retard the hydration of C_3A and related aluminate phases. However, the interaction between C_3A and a retarder is quite complex, and in some cases the early reactions may be accelerated, even though the overall hydration is retarded. An admixture also may be adsorbed into the early C_3A hydration products while they are being formed. The C_3A–admixture interactions are responsible for the following observations:

1. The effectiveness of a retarder depends on the C_3A content of the cement. (More retarder is removed from solution during the formation of the hydration products of C_3A, so that less is available to retard C_3S hydration.)
2. The effectiveness of a retarder is increased if its addition to the fresh concrete is delayed for a few minutes (see Figure 8.9). (Less retarder is removed from solution, since some hydration products have already formed before the admixture is added.)
3. A retarder may cause abnormal setting problems with particular cements: Both early stiffening and abnormal retardation of setting have been observed. Cements from the same mill have been found to behave differently with a water-reducing admixture, depending on the SO_3 content of the cement. (It appears that the aluminate/sulfate ratio can be thrown out of balance when the retarders change the rate of hydration of C_3A.)
4. An admixture may extend setting times, but not the time during which the concrete can be handled and placed. (Acceleration of the early reactions of C_3A can affect workability and may promote slump loss.)
5. A retarding admixture may influence the behavior of an expansive cement. (The rate of formation of ettringite is changed.)

In addition, the content of alkali oxides in a cement may determine the effectiveness of a retarder. It is not known exactly what causes this, but the presence of alkali

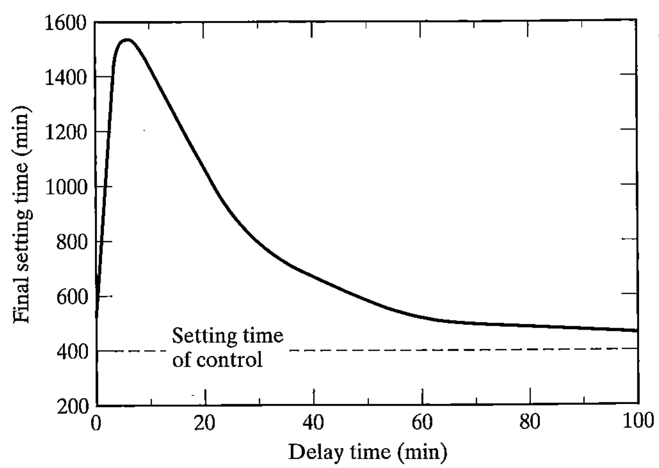


FIGURE 8.9
Effect of delayed addition of a retarding admixture on its retarding power.

sulfate will influence C_3A hydration. Also, it has been postulated that the alkalis may cause the breakdown of the admixture. Because retarders are so sensitive to cement composition, an admixture should be evaluated with the cement that is to be used on the job. An admixture–cement incompatibility usually can be eliminated by simply changing either cement or admixture.

Effect of Concrete Properties The influence of a set-retarding admixture on air entrainment should be considered, particularly if the admixture also has water-reducing properties. A retarder may also increase the rate of loss of workability in fresh concrete (slump loss), even when abnormal setting behavior does not occur. When this happens, the time during which the concrete can be handled and placed will be decreased, even though the setting time has been extended.

As mentioned earlier, retardation increases when the addition of a retarder to concrete is delayed a few minutes after the first addition of water. Retardation is increased rapidly up to about a 10-min delay (depending on admixture type and dose) and then slowly decreases (see Figure 8.9). Consequently, an additional dose of a water-reducing admixture can be used during retempering of fresh concrete without prolonging setting times unduly and without compromising the performance of the hardened concrete (which adding water would do).

Whenever a retarding admixture is used, some reduction in the one-day strength of the concrete should be anticipated, unless there is a concomitant reduction in *w/c* ratio. However, the strength should approach that of an unretarded concrete within eight days, unless an overdose has been used (see Figure 8.10). Retarding admixtures have been reported to increase ultimate compressive strength and, to a lesser extent, flexural strength. Although set-controlling admixtures have been reported to increase drying shrinkage and creep, the effects are not large and often depend on changes in mix design, time of hydration, and time of drying or loading. Laboratory work indicates that admixtures increase the rate of drying shrinkage and creep, but not the ultimate values.

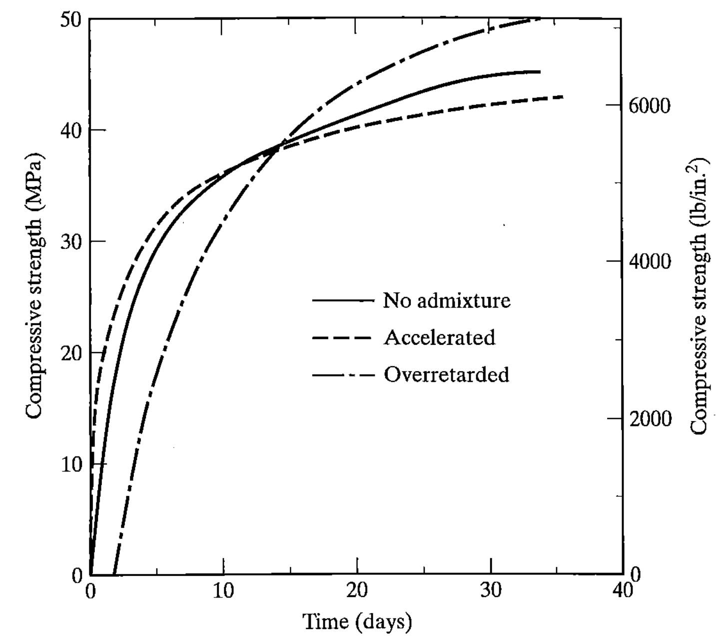


FIGURE 8.10
Effect of set-modifying admixtures on strength development of concrete.

Set-Accelerating Admixtures

When considering set-accelerating admixtures, it is important to distinguish between admixtures that accelerate the normal processes of setting and strength development and those that provide very rapid setting characteristics not normally associated with ordinary portland cements. Quick-setting admixtures provide setting times of only a few minutes and are used in shotcreting applications, for plugging leaks while under hydrostatic pressure, and for emergency repair in general where very rapid development of rigidity is required. Regular accelerators are used to speed construction by permitting earlier finishing of flatwork and earlier attainment of sufficient strength to allow removal of formwork and to carry construction loads. Accelerators are beneficial during winter concreting by partially overcoming the slower rate of hydration caused by low temperatures and shortening the period for which protection against damage by freezing is required. However, as will be discussed, accelerators can negatively impact the properties of the concrete and, therefore, should be used with caution.

Composition We can divide accelerating admixtures into three groups: (1) soluble inorganic salts, (2) soluble organic compounds, and (3) miscellaneous solid materials. Many soluble inorganic salts will accelerate the setting and hardening of concrete to some degree, calcium salts generally being the most effective. Calcium chloride is the most popular choice because it gives more acceleration at a particular rate of addition than other accelerators and is also reasonably inexpensive. In reinforced concrete, it has been replaced by calcium nitrite, which does not promote corrosion (see the discussion that follows). Soluble carbonates, aluminates, fluorides, and ferric salts have

quick-setting properties. Sodium carbonate and sodium aluminate are the most common ingredients of shotcreting admixture formulations used to promote quick setting.

A variety of organic compounds have accelerating properties (although many more act as retarders); triethanolamine, calcium formate, and calcium acetate account for most commercial uses. The former is commonly used in formulations of water-reducing admixtures to offset their retarding action. Although triethanolamine is listed as an accelerator, recent research shows that its reaction with portland cement is rather complex. It can cause retardation or flash setting, depending on the amount used.

Solid materials are not often used for acceleration. Calcium fluoroaluminate or calcium sulfoaluminate (Chapter 3) can be used as admixtures to obtain rapid-hardening characteristics. Additions of calcium aluminate cements cause portland cements to set rapidly, and concrete can be "seeded" by adding fully hydrated cement that has been finely ground during mixing to cause more rapid hydration. Finely divided carbonates (calcium or magnesium), silicate minerals, and silicas are reported to decrease setting times.

Mode of Action Conventional accelerators have exactly the opposite action that retarders have: they increase the rate of hydration of C_3S and the rate of strength development (see Figure 8.10). Generally, the induction period is shortened, and the rate of hydration during Stages 3 and 4 increased. This is the case with calcium salts. The addition of the admixture increases the rate at which calcium hydroxide crystals nucleate and grow. Organic accelerators are believed to act by increasing the rate of hydration of C_3S. Quicksetting admixtures cause flash setting of C_3A by promoting very rapid hydration on contact with water. Admixtures that affect C_3A hydration can also be expected to affect the expansion characteristics of expansive cements.

Effects on Concrete Properties Accelerators do not as a rule have adverse effects on air entrainment, although trial batches should be used to check this. It must be remembered that use of an accelerator will cut down the time during which the concrete can be handled and placed, so that reliable scheduling of construction operations is necessary. Accelerating admixtures may require some additional water if early stiffening occurs, but they can be formulated with a water-reducing admixture. Regular accelerators can be expected to increase one-day strengths, the increase depending on the type and dosage of the admixture and the temperature. An "optimum" dose of calcium chloride (2% by weight of cement)[2] will approximately double the one-day compressive strength of concrete (depending on the curing temperature). These increases diminish with time, and later strengths (at 28 days or more) are likely to be lower than the strength of concretes without an accelerating admixture (see Figure 8.10). This reduction in later strength is more pronounced when the initial accelerating effects are large. The extreme case is that of some quick-setting admixtures, which may have lower strengths at one day, even though some strength is obtained very rapidly.

Accelerating admixtures are said to increase drying shrinkage and creep, but, as discussed with retarders, it is the rate of deformation, rather than the ultimate value, that is affected. However, early shrinkage leads to high tensile stresses in restrained

[2] This is 2% of Type I calcium chloride (88% pure); see ASTM D 98 specifications for calcium chloride.

concrete, resulting in an increase in cracking. Tests have shown that the use of calcium chloride in concrete reduces its resistance to sulfate attack and aggravates the alkali–aggregate reaction if this is a potential problem. Quick-setting admixtures can be expected to reduce the durability of the concrete or mortar if they have an adverse effect on strength. Calcium chloride can discolor concrete and should, therefore, never be used in architectural concrete.

Corrosion of Reinforcement A major disadvantage attending the use of calcium chloride is its tendency to increase the rate of corrosion of metals embedded in concrete. In the distant past, additions of calcium chloride (up to 2% by weight of cement) were considered safe to use in ordinary reinforced concrete provided that an adequate cover of dense concrete was provided. However, the ACI Building Code places limits on the chloride content of concrete that preclude its use for both prestressed and reinforced concrete. Chloride-free accelerators should be used in such cases.

8.6 MISCELLANEOUS ADMIXTURES

There are many different admixtures used to produce special modifications to the behavior and performance of concrete and mortar. Only a few of the more important categories will be described here.

Viscosity Modifiers

High molecular weight, water-soluble polymers are used to raise the viscosity of water. Such compounds increase the cohesiveness of fresh concrete, reducing its tendency to segregate and bleed. These admixtures are helpful in improving the properties of lean concretes with low cement contents, concrete placed under water, and concretes or grouts that are placed by pumping. In the latter case, they reduce pumping pressures through improved lubricating properties, as well as reducing segregation tendencies. For this reason, they are often referred to as *pumping aids*. When compounds in this category are used to improve the cohesiveness of concrete being placed underwater, they are classified as *antiwashout admixtures*.

The materials commonly used are polyethylene oxides, cellulose ethers, alginates (from seaweed), natural gums, and polyacrylamides or polyvinyl alcohol. Other materials used are finely divided solids such as clays, lime, polymer emulsions, etc. The solid materials tend to reduce the strength of the concrete and are primarily used in grouts when strength is not of major importance.

Antifreezing Admixtures

When concrete is placed under very cold conditions, fresh concrete may still freeze before it has gained adequate strength, even if an accelerating admixture is used. To avoid this, admixtures have been developed to lower the freezing point of water, allowing the concrete to gain the required strength to resist the effects of freezing. They are intended for use where ambient temperatures remain consistently below freezing for long periods. Antifreeze compounds, such as high-molecular-weight alcohols (e.g., polyethylene glycol) can be added to lower the freezing point. High concentrations of chlorides, nitrates,

or nitrites (or mixtures of these) give smaller reductions in the freezing point of water, but accelerate the release of heat of hydration. Quite high doses of the chemicals are required (5–10% by weight of cement) to protect concrete down to −20°C. The admixtures must be formulated to give satisfactory set control and hardening.

Although popular in Russia, these admixtures have not been used extensively in the U.S. because the winter climate is not rigorous enough, except in parts of Alaska and other polar regions. Elsewhere, it is preferable to use artificial heating or to wait until the weather moderates sufficiently. Concrete can be kept from freezing down to −5°C (25°F) when properly proportioned using its heat of hydration and suitable insulation, to keep it from freezing. A conventional set accelerator can be used to speed up early hydration.

Expansion-Producing Admixtures

Admixtures that produce expansion in concrete are used to offset shrinkage that may occur in both fresh and hardened concrete. There are three major categories:

1. Gas-forming admixtures that control settlement and provide expansion prior to setting;
2. Admixtures that form expansive hydrates to provide expansion in the hardened state and prevent settlement due to bleeding of the fresh concrete;
3. Admixtures that control expansion only in the hardened state.

These admixtures are used extensively in grouting prestressing strands and anchoring bolts or cables and in the repair of concrete. Their use can ensure a good bond, reduce bleeding, and minimize drying shrinkage cracking.

Gas-Forming Admixtures Gas-forming admixtures are also used to produce lightweight cellular concrete for thermal insulation. The most common admixture is unpolished aluminum powder, which reacts at high pH to form hydrogen:

$$Al + 2OH^- + 2H_2O \rightarrow Al(OH)_4^- + H_2\uparrow \qquad (8.1)$$

Magnesium and zinc react similarly, but aluminum is most commonly used. The rate of hydrogen generation must be adjusted to give the desired volumetric expansion before setting occurs. It depends on the temperature, the pH of the fresh concrete (or grout), the reactivity of the powder, and the fluid consistency of the concrete (grout). Other gas-generating components are activated carbon or coke (releasing air), hydrogen peroxide (O_2), and compounds with an N–N bond in the molecule (N_2). Special organic foaming agents may be used to produce stable coarse foams, which are either blended into a concrete mix or produced during mixing.

Expansive Hydrates Two main chemical reactions are utilized by these admixtures. The first is the formation of ettringite from a mixture of 30% calcium sulfoaluminate (C_4A_3S), 50% $CaSO_4$, and 20% CaO, which is the basis of shrinkage-compensating cement. Admixtures based on this reaction can be used with portland cement in place of Type E-1(K) cement to produce shrinkage-compensating concrete. The second is the formation of $Ca(OH)_2$ from free lime of controlled reactivity (> 80% CaO). These formulations are sufficiently reactive to reduce bleeding in fresh concrete, but bulk expansion only begins when a rigid matrix is developed by the hydration of tricalcium silicate in the cement. The oxidation of iron to hydrated ferric oxide (rust) is a much slower reaction that occurs only after hardening has commenced. The rate of oxidation is controlled by the use of either corrosion-promoting or corrosion-inhibiting compounds.

Corrosion Inhibitors

These are admixtures that reduce the tendency for reinforcing steel to undergo corrosion in concrete. Such compounds probably do not inhibit the corrosion reactions completely, but reduce the rate of corrosion to a level that major damage to concrete will be avoided or at least greatly reduced. Corrosion inhibitors are classified according to how they interfere with the corrosion process, as summarized in Table 8.5. Anodic inhibitors are the most widely used. Calcium nitrite is sold as a nonchloride accelerator as well as a corrosion inhibitor. (The method by which it protects steel from corrosion is described in Chapter 18.) The use of sodium salts is not advisable if the aggregates used in the concrete are susceptible to alkali–aggregate reactions.

Alkali–Aggregate Reaction Inhibiting Admixtures

Lithium and barium salts can reduce the expansion and cracking associated with the alkali-silica reaction. It is thought that the conversion of the alkali–silica gels to insoluble lithium or barium salts may be the cause of this reduction. Lithium and ferric salts are effective in reducing expansion from the alkali–carbonate reaction. Although lithium salts are expensive, they are the most effective.

Shrinkage-Reducing Admixtures

The newest class of commercial chemical admixtures are those that reduce the amount of shrinkage that occurs as hardened concrete dries. The addition of certain water-soluble organic compounds to the fresh concrete lowers the shrinkage by 30–50%. The

TABLE 8.5 Types of Corrosion Inhibitors

Category	*Examples*	*Addition**	*Mode of Action*
Anodic inhibitors	Calcium nitrite	0.5–2	Promotes formation of a protective passive film.
	Sodium chromate	2–4	
	Sodium benzoate	6–8	
	Stannous chloride	2–4	
Cathodic inhibitors	NaOH or Na_2CO_3	2–4	Increases pH and lowers solubility of Fe^{2+}.
	Aniline derivatives (Aminobenzene)		Proton acceptors that adsorb on the cathode.
Mixed inhibitor	Aminobenzenethiol		Adsorbs at both anodic and cathodic sites. (Amino group adsorbs at cathode, thiol group at anode.)

*% of cement mass.

TABLE 8.6 Types of Damp-Proofing Admixtures

Chemical	*Method of Action*
Liquid fatty acids (stearic, oleic, caprylic, capric, etc.)	React with $Ca(OH)_2$ to form calcium salts
Butyl stearate emulsion	Reacts slowly with $Ca(OH)_2$ to form calcium stearate
Wax emulsion	Coalesces to hydrophobic layer
Emulsions of vegetable or animal fats	Form fat films
Calcium or aluminate stearates	Make cement paste hydrophobic

concrete can be post-treated with the same chemical, and this is a useful treatment for slabs on grade where moisture loss and its accompanying shrinkage occur only near the surface.

Damp-Proofing Admixtures

It is not possible to render concrete waterproof by the addition of an admixture. Since concrete is a porous material, water will penetrate concrete when under hydraulic pressure. The best option to reduce the penetration of water into and through concrete is to add a mineral admixture, use a low *w/cm* ratio, and cure well. However, concrete can be rendered more hydrophobic so that it will be better able to resist the penetration of rainwater or groundwater. The addition of hydrophobic compounds (see Table 8.6) is useful for this purpose, and they are called *damp-proofing admixtures*. These compounds act to make the surfaces of pores nonwetting (i.e., they increase the contact angle of water with the pore surface). This will increase the external force required to overcome the surface forces and penetrate the pore.

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PROBLEMS

8.1. What tests would you make to determine the suitability of an admixture for concrete?

8.2. What problems may arise when two or more admixtures are used simultaneously?

8.3. Why is it important to have accurate control of the batching of admixtures?

8.4. What are the effects of high ambient temperatures on the behavior of admixtures?
8.5. Not every material that acts as a detergent is suitable as an air-entraining agent for concrete. Why?
8.6. When can a water-reducing agent be used to advantage?
8.7. What are the potential disadvantages of superplasticizers?
8.8. Why are superplasticizers more effective than conventional water-reducing agents?
8.9. Are there any circumstances when you might want to use both an accelerator and a retarder?
8.10. Why can cement composition have an influence on the effectiveness of retarding admixtures?
8.11. What problems are associated with using $CaCl_2$ as an accelerating admixture? What is an alternative admixture?
8.12. A damp-proofing admixture gives a contact angle of 120° between water and the concrete surface. Calculate the resistance to the penetration of wind-driven rain into the concrete (in km/h of windspeed). The surface tension of water is 82 mJ/m^2 and the impact pressure of an average raindrop is 1.4 mJ/m^2 per km/h of windspeed. [Hint: Apply the Laplace equation $P = (-2\gamma\cos\theta)/r$.]

CHAPTER 9

Fresh Concrete

The properties of fresh concrete are important primarily because they affect the choice of equipment needed for handling and consolidation and because they may affect the properties of the hardened concrete. There are therefore two sets of criteria that we must consider when making concrete:

1. Short-term requirements, while the concrete is still in the plastic state, which are generally lumped together under the term "workability."
2. Long-term requirements of the hardened concrete, such as strength, durability, and volume stability.

Unfortunately, as will be seen when the problem of proportioning concrete is considered in Chapter 10, these two sets of requirements are not complementary, and a compromise is needed between them. In this chapter, only the short-term requirements for concrete are considered.

For hardened concrete to be of an acceptable quality for a given job, the fresh concrete must be capable of satisfying the following requirements:

1. It must be easily mixed and transported.
2. It must be uniform throughout a given batch and between batches.
3. It should have flow properties such that it is capable of filling completely the forms for which it was designed.
4. It must have the ability to be compacted fully without an excessive amount of energy being applied.
5. It must not segregate during placing and consolidation.
6. It must be capable of being finished properly, either against the forms or by means of troweling or other surface treatment.

9.1 WORKABILITY

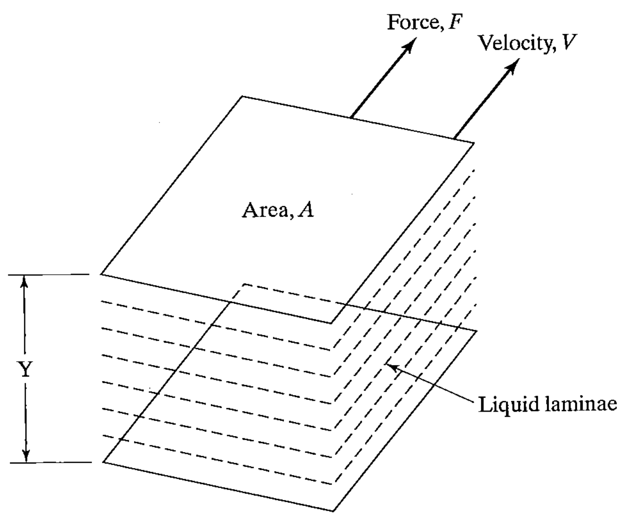
Definition of Workability

The aspects of the quality of fresh concrete just mentioned all have different requirements, so a number of terms are in common usage, each emphasizing a different facet of concrete behavior: consistency, flowability, mobility, pumpability, compactibility, finishability, and harshness. These terms are both subjective and qualitative; they mean different things to different people and are therefore not very useful. Here we use the term *workability* to represent all of the properties mentioned. To be a little more precise, workability is often defined in terms of the amount of mechanical work, or energy, required to produce full compaction of the concrete without segregation. This is a useful definition, since the final strength of the concrete is in large part a function of the amount of compaction; a small increase in void content (or decrease in relative density) will lead to a large decrease in strength.

Basic Principles of Rheology

Before discussing the various ways of measuring workability, it is helpful to examine a few basic principles of rheology, which is the science dealing with the deformation and flow of materials under stress. The simplest fluid is one that obeys Newton's law of viscous flow, which is illustrated in Figure 9.1. This law is derived by considering the shearing of adjacent layers of liquid, and it can be written

$$\tau = \eta\dot{\gamma} \tag{9.1}$$



Shear stress, $\tau = \dfrac{F}{A}$

Rate of shear, $\dot{\gamma} = \dfrac{dV}{dY}$

Coefficient of viscosity, $\eta = \dfrac{\tau}{\dot{\gamma}}$

FIGURE 9.1

Newton's law of viscous flow.

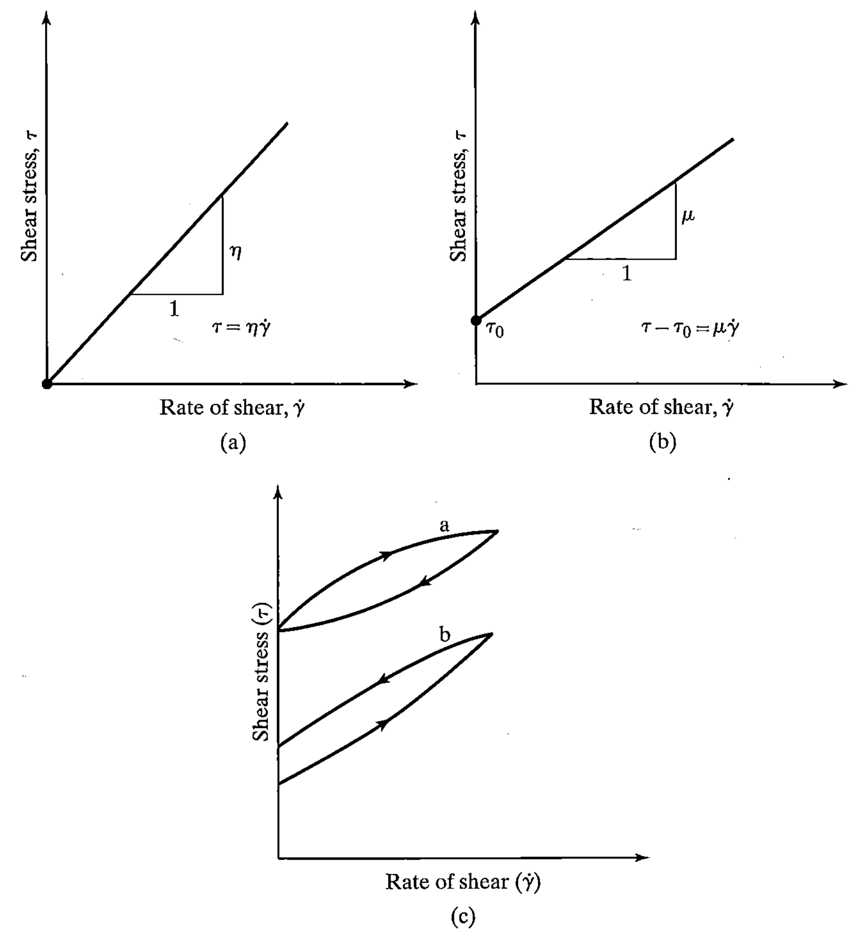


FIGURE 9.2

Rheological models: (a) Newtonian liquid; (b) Bingham model; (c) thixotropy.

where τ is the shear stress, η the coefficient of viscosity, and $\dot{\gamma}$ the rate of shear or the velocity gradient. From Eq. (9.1), it may be seen that a plot of the rate of shear vs. the shear stress will give a straight line passing through the origin, with a slope of $1/\eta$, as shown in Figure 9.2a. Therefore, if concrete could be approximated as a Newtonian fluid, a single measurement of a corresponding pair of values of τ and $\dot{\gamma}$ would serve to define the line. In other words, a "single-point" method of determining the workability would be sufficient.

For very dilute suspensions of a solid in a liquid, where there are no interparticle forces, Newton's equation holds. The effect of small increases in the amount of suspended solid is merely to increase the coefficient of viscosity (i.e., to decrease the slope of the line in Figure 9.2a). However, the Newtonian model breaks down for fluids in which the volume of suspended solids is large. Fresh concrete can be considered to be a very concentrated suspension. Typically, the ratio of the volume of solids to the volume of water would be about 4.5:1. For such materials, there are forces acting between the particles. This does not merely increase the viscosity (as in a Newtonian fluid), but

actually changes the type of flow behavior. In particular, the fresh concrete has a definite *shear strength*, τ_0, which must be exceeded before flow can occur. A common description of materials that exhibit this type of behavior is given by the *Bingham model*,

$$\tau - \tau_0 = \mu\dot{\gamma} \tag{9.2}$$

where τ_0 is the yield stress and μ is called the plastic viscosity. Such behavior is shown in Figure 9.2b. Clearly, for such a material, which is defined by two constants, τ_0 and μ, two pairs of values of τ and $\dot{\gamma}$ must be determined in order to define the straight line of Figure 9.2b. A single-point test would not be very useful in trying to describe such a fluid. There is now considerable evidence that the behavior of fresh concrete can be reasonably approximated by the Bingham model. Thus, the single-point tests of workability currently in use do not adequately quantify the behavior of the material from the point of view of rheology.

One other term that must be defined is *thixotropy*. A thixotropic (or shear thinning) material is one that undergoes "a decrease of the apparent viscosity under shear stress, followed by a gradual recovery when the stress is removed. The effect is time dependent." Flow curves of a thixotropic fluid are shown in Figure 9.2c. A truly thixotropic fluid exhibits the fully reversible behavior shown in curve (a). If the behavior is not completely reversible (curve b), the material is considered to be "pseudo-thixotropic." Such behavior is due to interparticle attraction and weak bonding. Concrete will exhibit thixotropic behavior as well; this can be important when considering the placing and vibration of concrete.

A number of devices have been developed in recent years to try to measure the true rheological properties of concrete. In some of these, an impeller of some particular geometry is lowered into a sample of fresh concrete, as shown schematically in Figure 9.3. As the rotational speed of the impeller is increased, the torque in the vertical shaft increases, and a plot of impeller speed versus torque is obtained. From these data, the flow resistance of the concrete (which is related to the yield stress, τ_0, Figure 9.2) and the torque viscosity (which is related to the plastic viscosity, μ, Figure 9.2) can be calculated.

Other commercially available devices are of the form of coaxial-cylinders viscometers, in which the torque on the fixed (inner) cylinder is measured and plotted against the speed of the rotating (outer) cylinder, as shown schematically in Figure 9.4. Again, the rheological properties of the concrete can be obtained from the graph of torque vs. rotational velocity.

However, as yet, none of these tests has been standardized, and the agreement in results amongst the various devices is only fair, at best. While it is likely that the use of rheological tests to measure the properties of fresh concrete will increase in the future, particularly for special concretes (very high strength, flowing, self-leveling), the existing tests are now primarily suitable only for laboratory research.

Factors Affecting Workability

The workability of concrete is affected by a number of factors: water content of the mix, mix proportions, aggregate properties, time, temperature, characteristics of the cement, and admixtures.

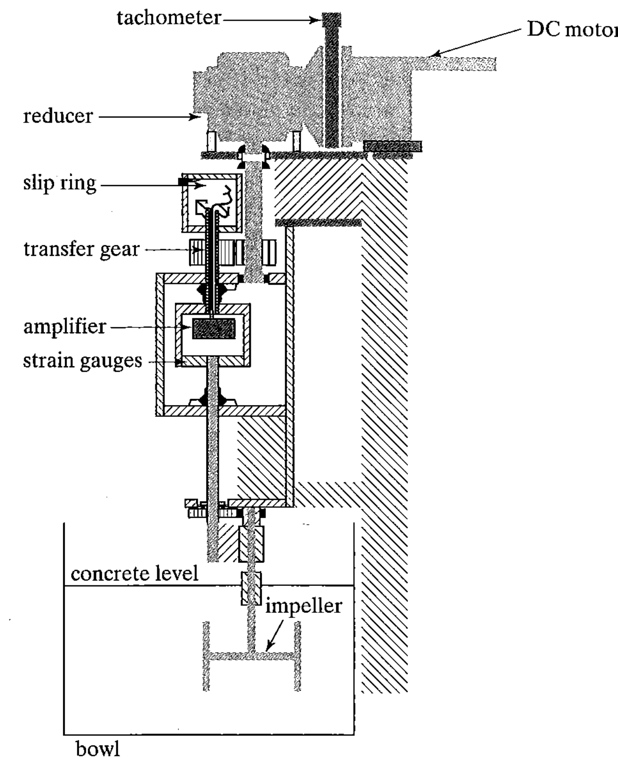


FIGURE 9.3

Schematic diagram of a concrete rheometer.

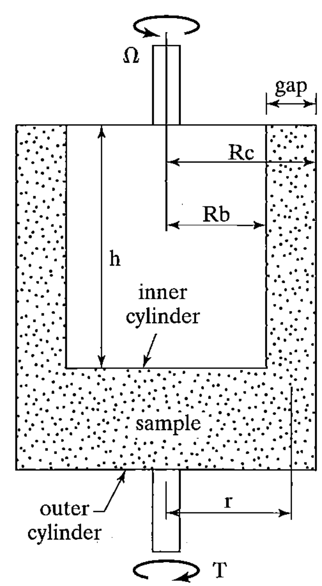


FIGURE 9.4

Schematic diagram of a coaxial cylinders viscometer.

Water Content of the Mix The single most important factor governing the workability of concrete is the water content. Increasing the amount of water will increase the ease with which concrete flows and can be compacted. However, apart from reducing the strength, increased water may lead to segregation (settling of the coarse aggregate) and to bleeding. In general, any collection of particles requires a certain amount of water to achieve plasticity so that it can be "worked." First, there must be enough water to adsorb on the particle surfaces. Then, water must fill the spaces between particles; additional water "lubricates" the particles by separating them with a water film. From this, it follows that finer particles, which have a higher specific surface area, require more water. On the other hand, without some minimum quantity of fine material, the concrete cannot exhibit plasticity. Thus, the water content of the mix cannot be considered in isolation from the aggregate grading. For optimum workability, finer aggregate gradings require high water contents.

Influence of Aggregate When considering the effect of aggregates on workability, two factors are important: the amount of aggregate and the relative proportions of fine and coarse aggregate. For a constant *w/c* ratio, an increase in the aggregate/cement ratio will decrease the workability; also, more cement is needed when finer aggregate gradings are used. A deficiency in fine aggregate results in a mix that is harsh, prone to segregation, and difficult to finish. (A harsh mix is one that lacks the desired consistency because of a deficiency in mortar or aggregate fines.) On the other hand, an excess of fine aggregate ("oversanded") will lead to a rather more permeable and less economical concrete, although the mixture will be easily workable.

However, it is not sufficient merely to look at the ratio of coarse/fine aggregate. Different sands will behave differently because of differences in particle-size distribution. This is generally expressed in terms of the fineness modulus, which was described in Chapter 7.

The shape and texture of aggregate particles can also affect the workability. As a general rule, the more nearly spherical the particles, the more workable the resulting concrete will be. This is due partly to the fact the spherical particles will act as "ball bearings" while angular particles will have more mechanical interlock and will therefore need more work to overcome the resulting internal friction. Spherical particles will have a lower surface-to-volume ratio, and less mortar will be needed to coat the particles, leaving more to provide "workability." When flat or elongated particles are contained in the coarse aggregate, the quantities of sand, cement, and water must be increased. Also, smooth particles will tend to be more workable than rough ones, such as are obtained when crushed rock is used as aggregate.

The porosity of the aggregates may also affect workability. If the aggregate can absorb a great deal of water, less will be available to provide workability. Thus, it may be necessary to distinguish between the *total* water content (which may include absorbed water) and the *free* water content (which is available to provide workability and which determines the *w/c* ratio).

Time and Temperature There is considerable evidence that as the ambient temperature increases, the workability decreases, as shown in Figure 9.5, since higher temperatures will increase both the evaporation rate and the hydration rate. This suggests that

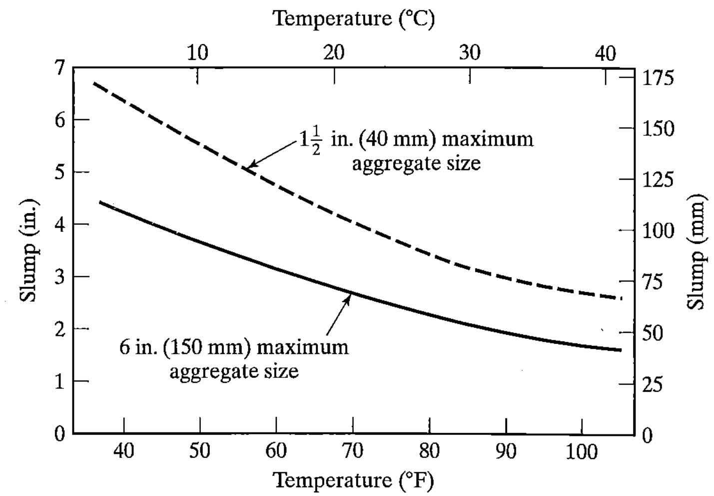


FIGURE 9.5

Relationship between slump and temperature of concrete made with two maximum sizes of aggregate. Each curve represents the average obtained with twelve different cements. [Adapted from *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975).]

more water may have to be used in very warm weather to maintain the same workability. Also, the quantity of water required to change the slump by some given amount increases as the temperature increases. On the other hand, there is also some evidence that, at least for short times, workability itself is not affected by temperature. Thus, where temperature may be a factor, field tests should be carried out with the specific materials in order to determine the temperature effects for a given job. The relationship between temperature and workability is discussed at greater length in Chapter 11.

Loss of Workability During the period of fluidity, there will be a steady decrease in workability with time, caused partially by the hydration of C_3S and C_3A, which continues slowly even during the dormant period, and partially by loss of water from the concrete through evaporation or absorption. In addition, particle interactions change because of the presence of hydration products on their surfaces. This "slump loss" is approximately linear with time, although it is greatest in the first $\frac{1}{2}$ to 1 h after mixing. Since the slump at the time of placement is of greatest importance, the slump loss of concrete must be considered when selecting mix proportions. Slump loss is increased when concrete temperatures are higher and also when accelerating admixtures are used; in both cases, setting times are decreased. Accelerated slump loss can occur when water-reducing and set-retarding admixtures are used, even when the setting times are unchanged or even increased. The rate of loss of workability is less for lean (low-cement content) mixes and when high *w/c* ratios are used. The brand of cement is also important (Figure 9.6), since two cements of the same type can have different compound compositions.

If the concrete mix is improperly designed, it may be necessary to remix the concrete with additional water ("retempering") just prior to placing in order to restore workability. Retempering increases the *w/c* ratio and may be highly detrimental to the properties of the concrete unless additional cement is also added. Therefore, retempering is a practice that should be strongly discouraged.

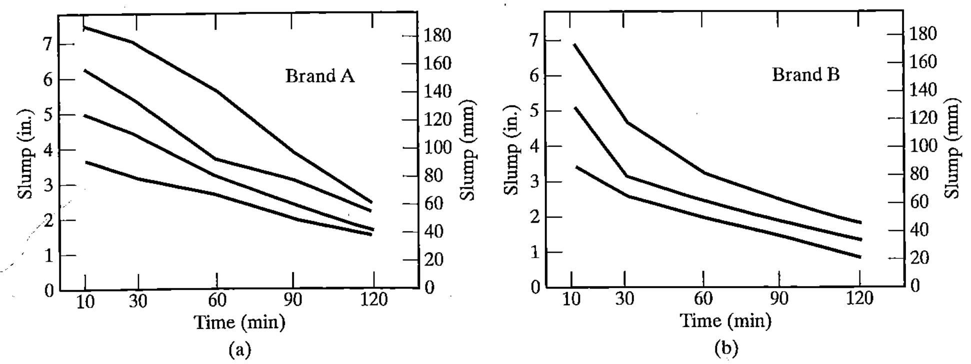


FIGURE 9.6

Concrete slump loss at 21°C for two different brands of Type I cement. [From R. W. Previte, *Journal of the American Concrete Institute*, Vol. 74, No. 8, p. 361 (1977).]

Cement Characteristics The cement characteristics are much less important in determining workability than are the aggregate properties. However, the increased fineness of Type III (high early strength) cements will reduce workability at a given *w/c* ratio, as these cements have a higher water requirement because of their higher specific surface area and because they also hydrate more rapidly. It has been found that if cement is delivered to the job hot (particularly if higher than 100°C), there may be a considerable loss of workability, due to more rapid hydration and evaporation of water. In addition, at temperatures higher than 60 to 80°C, flash set may occur.

Admixtures The different types of admixtures have already been discussed in detail in Chapters 5 and 8. Mineral admixtures are often used to supply additional fine material to harsh mixes; when they are used simply to *replace* part of the cement, they may have little effect on workability. Air-entraining, water-reducing, and set-retarding admixtures will all improve the workability. However, it should be noted that chemical admixtures react differently with different cements and aggregates and can in some circumstances aggravate the loss of workability. In particular, when using the high quantities of superplasticizers required for some high-performance concrete mixes, the problem of *cement–superplasticizer compatibility* must be addressed. Since we cannot yet predict exactly how a given superplasticizer will react with a particular cement, this must be done empirically, through laboratory and field trials.

Segregation and Bleeding

Segregation refers to a separation of the components of fresh concrete, resulting in a nonuniform mix. In general, this means some separation of the coarse aggregate from the mortar. This separation can be of two types: either the settling of heavy particles to the bottom of the fresh concrete, or a separation of the coarse aggregate from the body of the concrete, generally due to improper placing or vibration. Although there are no quantitative tests for segregation, it can be seen quite clearly when it does

occur. The factors that contribute to increased segregation have been listed by Popovics[1] as follows:

1. Larger maximum particle size (over 25 mm) and proportion of the large particles.
2. A high specific gravity of the coarse aggregate compared to that of the fine aggregate.
3. A decreased amount of fines (sand or cement).
4. Changes in the particle shape away from smooth, well-rounded particles to odd-shaped, rough particles.
5. Mixes that are either too wet or too dry.

The use of finely divided mineral admixtures or air-entraining agents reduces the tendency toward segregation, but careful handling and placing methods are more important. The correct techniques of handling concrete to prevent segregation will be described in more detail in Chapter 11.

Bleeding may be defined as the upward movement of water after concrete has been consolidated, but before it has set. Water, being the lightest component, segregates from the rest of the mix, and thus bleeding is a special form of segregation. Water on the surface of concrete is the most common manifestation of bleeding. Bleeding is generally caused by the fact that as the aggregate particles settle within the mass of fresh concrete, they are unable to hold all the mixing water. Some bleeding is normal for good concrete; it results in a small amount of uniform seepage over the entire surface.

On the other hand, bleeding may occur through distinct, localized channels, and small "craters" may form at the mouth of each channel. The upper layer of the concrete may become rich in cement paste, which has a *w/c* ratio that is too high. This leads to weakness, porosity, and a lack of durability. Water pockets may form under large aggregate particles or reinforcing bars, leaving weak zones in the concrete and reducing bond (Figure 9.7). If the bleed water evaporates more quickly than the bleeding rates, which often occurs in hot, dry weather, plastic shrinkage cracks will form (Chapter 16). Sometimes, a scum of fine particles may be carried to the surface, creating a weak and nondurable surface, or salts may crystallize at the surface, leading to the formation of $CaCO_3$. This is referred to as *laitance*. If this occurs at the top of a lift, poor bond to the next lift will be the result. If it occurs at the top of a flat slab, the surface will be prone to *dusting*; that is, a dry, powdery layer will appear on the surface of the hardened concrete, and this will be a permanent feature of the concrete. Deterioration is then a combination of laitance and plastic shrinkage. Thus, laitance should always be removed by brushing and washing the surface.

Bleeding can be reduced in a number of ways:

1. By increasing cement fineness or by using pozzolans or other finely divided mineral admixtures.

[1] S. Popovics, in *Proceedings of a RILEM Seminar, March 22–24, 1973, Leeds*, Vol. 3, The University, Leeds, pp. 6.1-1–6.1-37 (1973).

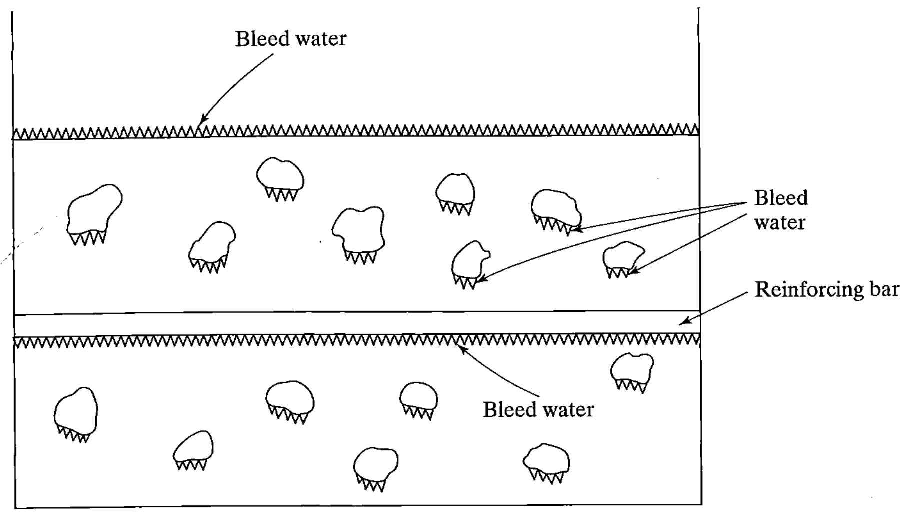


FIGURE 9.7
Different types of bleeding in concrete.

2. By increasing the rate of hydration of the cement by using cements with high-alkali contents or high-C_3A contents (which may, however, have other undesirable effects) or by using $CaCl_2$ as an admixture (which may also be undesirable).
3. Through air entrainment, which is very effective.
4. By reducing the water content, if this can be done while maintaining an acceptable workability.

Both the rate of bleeding and the total bleeding capacity can be measured using the procedures outlined in ASTM C 232. This method involves consolidating a sample of concrete in a container with an inside diameter of 254 mm (10 in.) and a height of 279 mm (11 in.). The container is then covered to prevent evaporation. The bleed water rising to the surface is carefully drawn off every 10 min for the first 40 min, and every 30 min thereafter until the bleeding has stopped. The total bleeding and rate of bleeding may then be determined. Two slightly different procedures, one for concrete consolidated by tamping and one for concrete consolidated by vibration, are outlined; these two types of consolidation will generally lead to different results.

9.2 MEASUREMENT OF WORKABILITY

As we have seen, the term "workability" is applied to a number of different properties of fresh concrete that are affected by a large number of factors. It has been suggested that workability should measure at least three separate concrete properties:

1. Compactibility, the ease with which concrete can be compacted and air voids removed.

2. Mobility, the ease with which concrete can flow into forms, around steel, and be remolded.
3. Stability, the ability for concrete to remain a stable, coherent, homogeneous mass during handling and vibration without the constituents segregating.

A large number of workability tests have been proposed over the years, almost all of them completely empirical. Only a few of these tests have been incorporated into standards; the rest have received very limited use. None of the available standardized tests measures workability in terms of fundamental properties of fresh concrete. In addition, they are all single-point tests and therefore cannot be expected to describe the properties of a material such as concrete, which appears to follow a Bingham model (Section 9.1). The available tests cannot even easily be compared to one another, since they tend to measure somewhat different properties of the concrete.

Nevertheless, it *is* important to have some measure of the workability of concrete. Thus, until some better test procedures are developed, we must make do with one or more of the available test methods. Most of the available methods do, at least, provide information as to *variations* in workability for a given mix and are therefore useful as quality-control measures. It might also be noted that none of the available test methods can be used over the whole range of very wet to very dry concrete; any individual test method can only be used over a relatively narrow range of workabilities.

The different measures of workability that have been developed over the years will be described under the following categories: (1) subjective assessment, (2) slump test, (3) compaction tests, (4) flow tests, (5) remolding tests, and (6) miscellaneous tests.

Subjective Assessment

The subjective assessment of fresh concrete (by an experienced worker) is, of course, the oldest "measure" of workability and is very widely used, at least as an adjunct to more quantitative tests. Some engineers still prefer to use their own judgment, based on the way the concrete behaves in the mixer and while it is being placed. Concrete may be described as being of high, medium, or low workability or as being wet, dry, plastic, and so on. Unfortunately, these terms mean different things to different people, so they may not be very helpful. Pity the poor engineer who had to work with this description of workability taken from an old textbook: "The mortar was wet enough to quake like liver under moderate ramming."[2]

Slump Test

The slump test is by far the oldest and the most widely used test of workability. It first appeared as an ASTM Standard in 1922 and now is described in ASTM C 143. The apparatus for this very simple test consists primarily of a hollow mold in the form of a frustrum of a cone, with the dimensions shown in Figure 9.8. The mold is filled with concrete in three layers of equal volume; each layer is rodded 25 times with a 16-mm ($5/8$-in.) diameter steel rod. The mold is then lifted away vertically, and the slump is

[2]L. C. Sabin, *Cement and Concrete*, Archibald Constable and Co., London, 1905.

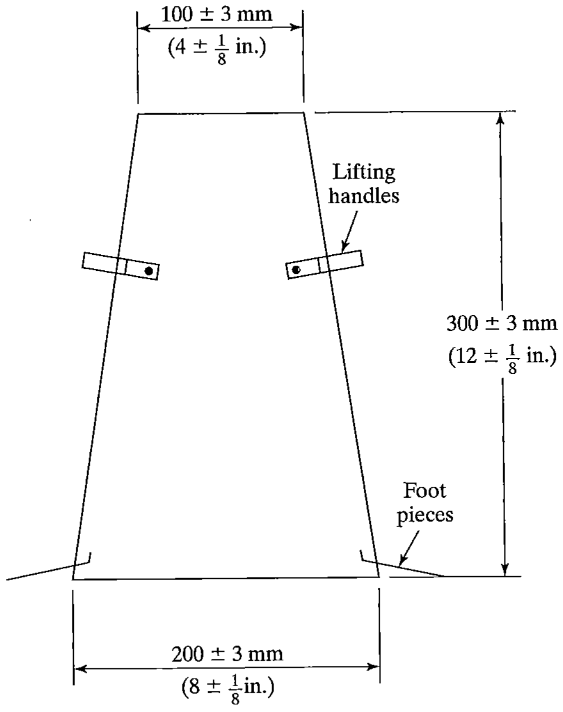


FIGURE 9.8

Slump cone.

measured by determining the difference between the height of the mold and the height of the concrete over the original center of the base of the specimen. The maximum time from first placement of concrete to lifting of the cone is $2\frac{1}{2}$ min. If a distinct shearing off of concrete from one side of the cone occurs, the test is disregarded and a new one carried out. If such shearing occurs consistently, this indicates that the test is not suitable for that particular concrete mix.

The slump test may be considered to be a measure of the shear resistance of concrete to flowing under its own weight. Depending on the mix, three distinct types of slump may occur, as shown in Figure 9.9: "*True*" *slump* consists of a general subsidence of the mass, without any breaking up. *Shear slump* often indicates a lack of cohesion; it tends to occur in harsh mixes or in mixes prone to segregation. Shear slump may indicate that the concrete is not suitable for placement. However, normal mixes are sometimes found to display shear slump. *Collapse slump* generally indicates a lean, harsh, or more likely, a very wet mix. On the other hand, many modern high-performance concretes, which are specially designed for a slump of about 200 mm (8 in.), display this type of slump behavior.

There are a number of difficulties associated with the slump test. The test is completely empirical and is not related to our earlier definition of workability, which involved a measure of the amount of energy required to compact concrete. With different aggregates or mix properties, the same slump can be measured for very different concrete consistencies. This is illustrated in Figure 9.10; in the views at the right, the slumped

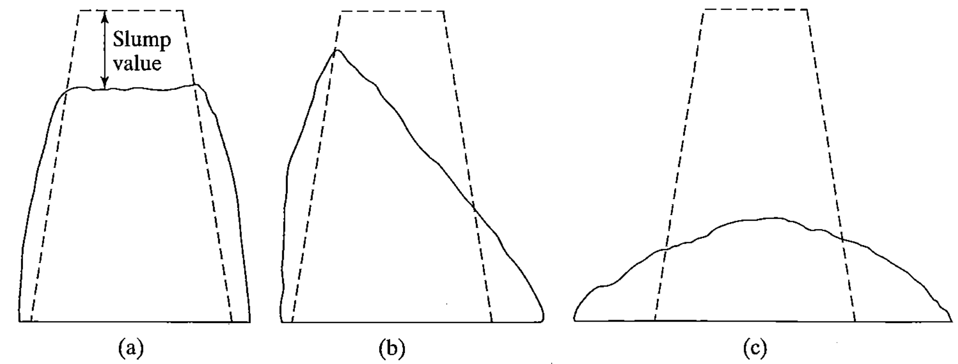


FIGURE 9.9

Types of slump of concrete: (a) true slump; (b) shear slump; (c) collapse slump.

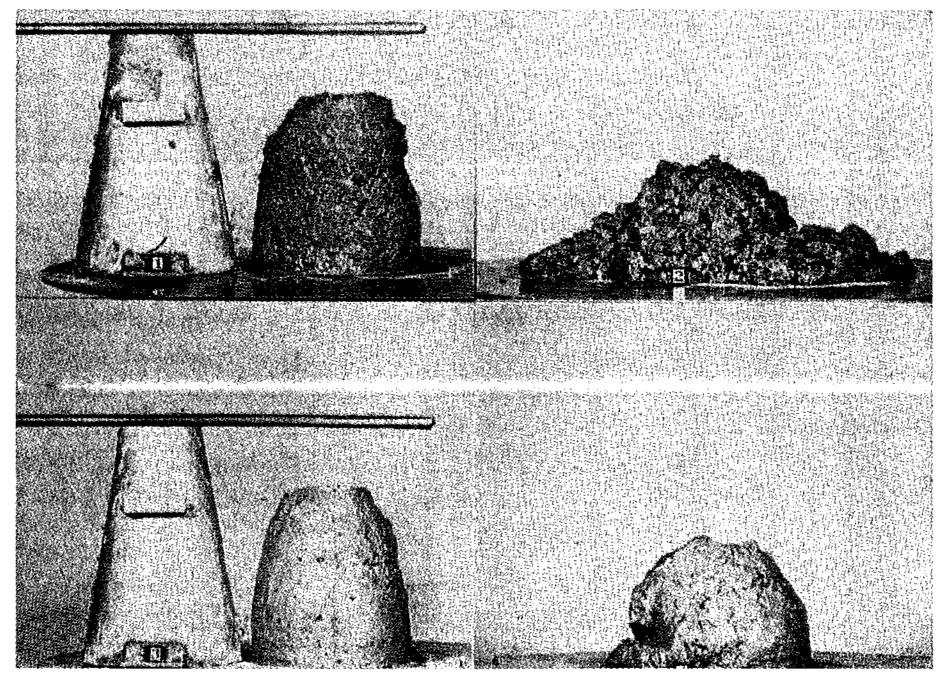


FIGURE 9.10

Slump test for consistency. By tapping the side of a slump specimen with the tamping rod (see the views at the right), additional information as to the workability of the concrete is obtained. [From *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975).]

concrete has been gently tapped on the side with the rod (according to the U.S. Bureau of Reclamation procedure) to provide additional information on workability. The upper concrete is a very harsh mix, with a minimum of fines and water. The lower concrete is

plastic and cohesive. Clearly, these two concretes could not both be used for the same purposes, even though their slump values are the same. In addition, the slump test cannot differentiate between different low-workability concretes, which may all give "no slump." Concretes with slumps less than 25 mm (1 in.) should be tested by another procedure, preferably one involving vibration, since vibration will be needed to consolidate such stiff concrete. Although the test is reasonably reproducible with a skilled operator, it is quite sensitive to variations in test procedure, and it is not uncommon for different operators to achieve values more than 25 mm (1 in.) apart for the same concrete. As indicated earlier when describing the workability–time relationships, the slump may vary considerably depending on how long after mixing the test is carried out.

In spite of these limitations, slump tests can provide useful information. In general, concretes of similar slump *can* be used the same way. More important, the slump test is a valuable quality-control tool. Changes in slump on a given job generally indicate that a change has occurred in the aggregates or in the amount of water or admixture being used. This should provide a warning that something is happening to the mix so that remedial action can be taken if necessary.

Compaction Tests

The strength of concrete is approximately proportional to the relative density. A test to measure the compactibility of concrete for a given amount of work more closely approaches our earlier definition of workability and can provide a very useful measure of concrete properties. A number of different compaction tests have been proposed, the most common being the compacting factor test.

Compacting Factor Test The compacting factor test was developed in Great Britain in 1947. The apparatus is shown in Figure 9.11. The upper hopper is completely filled with concrete, which is then successively dropped into the lower hopper and then into the cylindrical mold. The excess concrete is struck off, and the compacting factor is defined as the ratio of the concrete in the cylinder to the same concrete fully compacted in the cylinder.

Apart from the fact that the apparatus is not very suitable for field use, it has been found that some mixes stick to the sides of the hoppers (particularly when air-entrained) and must be rodded through. It has also been found that mixes with the same compacting factor do not necessarily require the same amount of work for compaction. Although some authorities feel that the test is suitable for very dry mixes (it is one of the tests referred to in ACI 211.3R, Guide for Selecting Proportions for No-Slump Concrete), others feel that it should be used only for "standard" mixes. Moreover, as the maximum size of aggregate increases, the size of the apparatus must be increased, making it impractical for aggregates greater than 40 mm ($1\frac{1}{2}$ in.) in size.

Flow Tests

Flow tests measure the ability of a concrete to flow under jolting or continuous vibration and provide information as to the tendency for segregation. A number of tests are

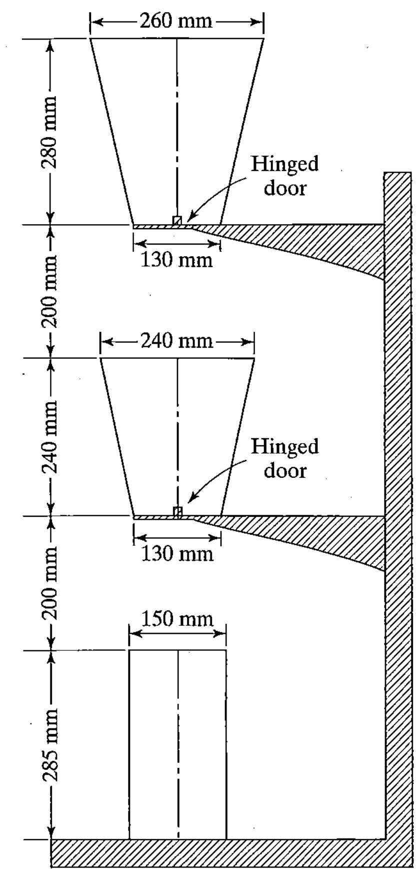


FIGURE 9.11
Compacting factor apparatus (British Standard 1881).

available, but there are none recognized by ASTM. However, until 1973, under ASTM C 124, a flow-table test was described that is still sometimes used. In this test, a sample of concrete is cast in the form of a frustrum of a cone, 254 mm (10 in.) in diameter at the base, 171 mm (6.75 in.) in diameter at the top, and 127 mm (5 in.) high. This is done on a drop table, which is then dropped 15 times in 15 s through a height of 12.7 mm ($\frac{1}{2}$ in.) by revolving a cam shaft. The flow is defined as the increase in diameter expressed as a percentage of the original diameter. Again, concretes with the same flow may have quite different workabilities in the field. Similar tests are still in use in parts of Europe. For example, a flow test for concrete consistency is described in the German DIN 1048, Part 1, Chapter 3.1.2.

Remolding Tests

Remolding tests were developed to measure the work required to cause concrete not only to flow, but also to conform to a new shape. They are intended to try to simulate, in the laboratory, actual field conditions. A number of tests were developed, but only the Vebe test is used to any extent.

Vebe Test The Vebe consistometer was developed in 1940 and is probably the most suitable test for determining differences in consistency of very dry mixes. There are three versions of the test: The first version (Figure 9.12) is widely used in Europe. It is, however, only applicable to concretes with a maximum size of aggregate of less than 40 mm ($1\frac{1}{2}$ in.). A standard slump cone is cast, the mold removed, and a transparent disk placed on top of the cone. This is then vibrated at a controlled frequency and amplitude until the lower surface of the transparent disk is completely covered with grout. The time in seconds for this to occur is the Vebe time. The test is probably most suitable for concretes with Vebe times from 5 to 30 s. The only real difficulty with the test is that the wetting of the disk with mortar is not uniform, and it may be difficult to pick out the end point of the test.

The second and third versions of the test are designed to evaluate the consistency of roller-compacted concrete (see Chapter 11) and are described in ASTM C 1170. In Test Method A, used for concrete of "very stiff to extremely dry consistency," the cylindrical container shown in Figure 9.12 is filled with concrete. A 22.7-kg (50-lb) weight is placed on top of a plastic base plate with a diameter of 229 ± 3 mm ($9 \pm \frac{1}{8}$ in.) centered within the container. The apparatus is vibrated until mortar fills the annular space between the outer edge of the surcharge and the mold wall. The time required is identified as the Vebe consistency time, Test Method A. A maximum vibration time of 2 min is used, and if the space has not been filled, the condition of the concrete is noted. Test Method B, used for concrete of "stiff to very stiff consistency," is similar to Method A, but without the surcharge. If the total Vebe time is less than 2 min, vibration is continued for a total of 2 min to obtain the Vebe density. A maximum aggregate size of 50 mm

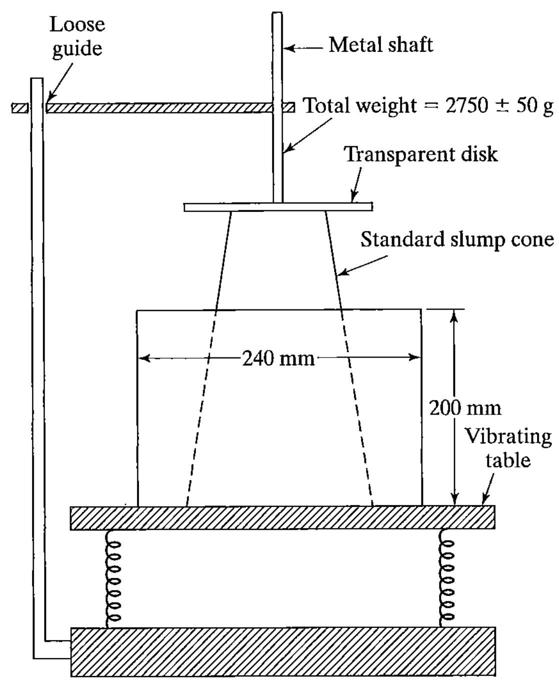


FIGURE 9.12
Vebe apparatus.

(2 in.) is used for both procedures. For concrete with larger aggregate, wet sieving is used to reduce the maximum aggregate size. The tests described in ASTM C 1170 are the test procedures preferred in ACI 211.3R.

Thaulow Drop Table This test is also suggested for no-slump concrete in ACI 211.3R. It is similar to the Vebe test in that it involves changing the shape of a sample of concrete from a slump cone to a cylinder. A drop table is used and the number of 10-mm (0.4-in.) drops to achieve this remolding is counted.

Miscellaneous Tests

One recent workability test that was adopted as an ASTM Standard in 1997 (ASTM C 1362), and that is also referred to as a "flow" test is shown schematically in Figure 9.13. The apparatus consists of a partially perforated outer hollow tube and an inner, calibrated floating rod. The apparatus is inserted into fresh concrete, and the amount of concrete that flows into the hollow tube in a period of 40 s is measured. With increasing fluidity (workability), a higher flow reading is obtained.

Other Tests

Over the years, many other tests have been proposed to measure concrete workability. These include:

1. Deformation tests in which the amount of work required to deform the concrete in a mold is measured.
2. Drop tests where the cohesiveness of the concrete is measured by seeing how much it segregates on being dropped.

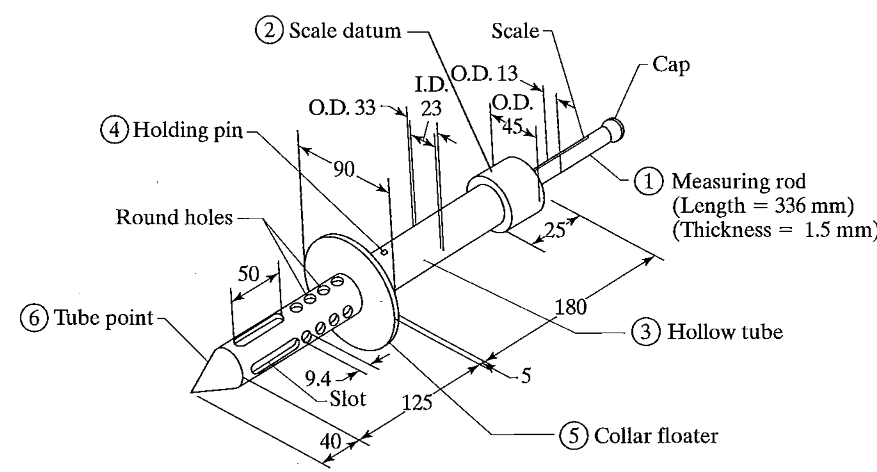


FIGURE 9.13
Apparatus for Flow Test. (Copyright ASTM INTERNATIONAL. Reprinted with permission.)

3. Mixer tests for which the amount of work required to mix the concrete in a concrete mixer is measured, most commonly by measuring the power required to turn the mixer.

Summary

1. All of the tests described are empirical and do not measure in any fundamental way the rheological properties of concrete.
2. None of the tests will work for all concretes, and they may yield the same values for concretes of quite different workabilities.
3. Their primary usefulness is as a quality-control measure *for a given concrete mix*; changes in the measurement may then indicate some change in the mix.

9.3 SETTING OF CONCRETE

Setting is defined as the onset of rigidity in fresh concrete. It is distinct from *hardening*, which describes the development of useful and measurable strength. Setting precedes hardening, but it should be emphasized that both are gradual changes that are controlled by the continuing hydration of the cement. We can view setting as a transitional period between states of true fluidity and true rigidity. The penetration tests used to measure the times of setting (which are described in Chapter 3) are purely arbitrary measurements. Figure 9.14 shows that initial set and final set, as measured by ASTM C

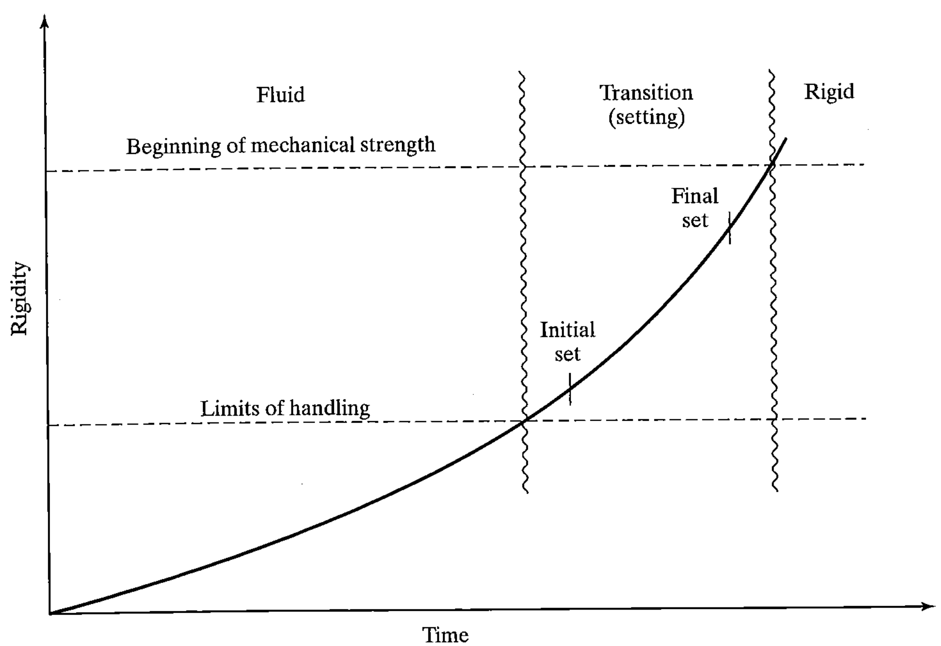


FIGURE 9.14
Process of setting and hardening.

403, do not correspond exactly to any specific change in concrete properties, although it is useful to consider that initial set represents approximately the time at which fresh concrete can no longer be properly handled and placed, while final set approximates the time at which hardening begins. Fresh concrete will have lost measurable slump prior to initial set, while measurable strength will be achieved sometime after final set.

Effect of Hydration on Setting

Role of C_3S It should be remembered that the cement hydration begins as soon as water is added at the mixer. Setting is controlled primarily by the hydration of C_3S. The period of fluidity corresponds to the induction period (Stage 2) of C_3S hydration (Chapter 4). Setting occurs when the induction period is terminated and rapid hydration of C_3S occurs in Stage 3. Initial set corresponds approximately to the beginning of Stage 3 and final set to its midpoint. Thus, initial set is marked by the beginning of a rapid temperature rise of the concrete, which will reach a maximum rate at final set. Setting is also accompanied by a decrease in electrical conductivity and an increase in the velocity of sound waves propagating through the paste. Measurements of either of these properties could form the basis of an adequate test for setting.

Role of C_3A and Gypsum When using ordinary portland cements (the five ASTM types), C_3A plays a relatively minor role in determining setting behavior, except in the cases of abnormal set discussed shortly. Gypsum is always interground with modern cements; it reacts with C_3A to help control the setting time. When large amounts of ettringite are formed rapidly, the aluminate phase will decrease the time to initial and final set, as in the case of expansive cement. In the extreme case (e.g., regulated-set cements), ettringite formation entirely controls setting.

Abnormal Setting Behavior

Abnormal setting of concrete was a troublesome problem in earlier times, but is much rarer now. The different types of abnormal setting behavior are shown in Figure 9.15. They are most likely to be encountered under certain conditions when an admixture is used (usually a set-retarding admixture). Two major types of setting problems may be encountered; *false set* and *flash set.*

False Set A concrete may stiffen rapidly a short time after mixing is completed (see Figure 9.15). Fluidity is restored by remixing, and the concrete will then set normally; thus, false set has more nuisance value than anything else. This phenomenon is sometimes called *plaster set* because it is most often caused by crystallization of gypsum. When gypsum is interground with clinker, the material in the grinding mill can get quite hot because of the high energy input during grinding. The temperature can rise high enough (~120°C or ~250°F) to cause the gypsum to partially dehydrate to calcium sulfate hemihydrate (plaster) according to the first part of Eq. (3.5) (Section 3.4). This may also occur if the gypsum is added to hot clinker from the kiln. When water is added, the hemihydrate will rehydrate back to gypsum and form a rigid crystalline matrix. However, because there are only small amounts of plaster in the concrete, very little strength

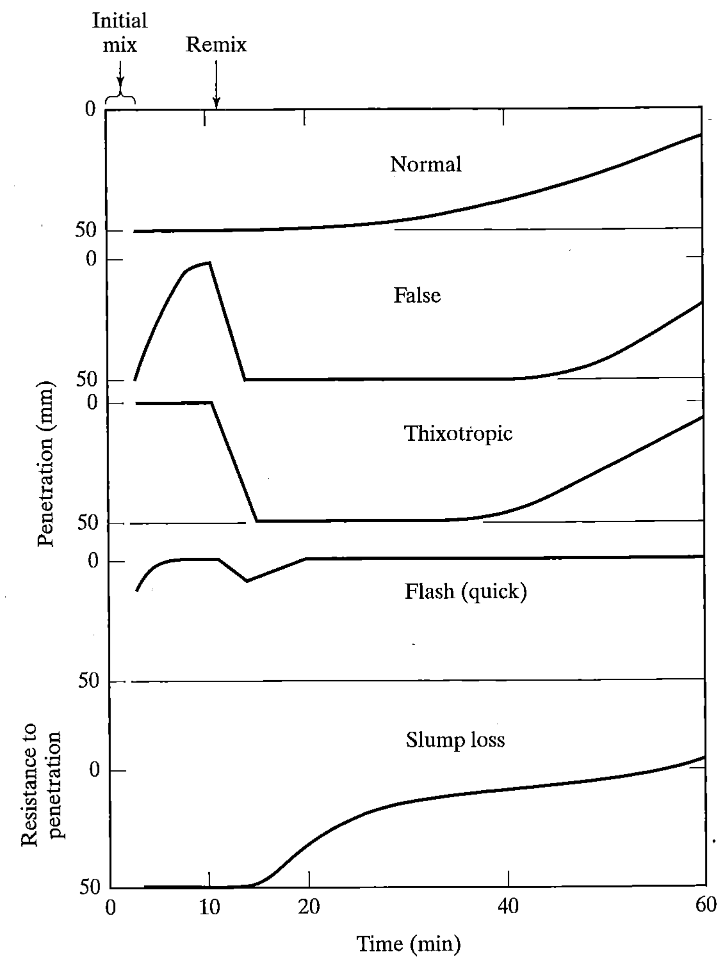


FIGURE 9.15

Diagrammatic sketches of different types of set of cement mortar and slump loss of concrete. [Adapted from G. L. Kalousek, *Abnormal Set of Portland Cement. Causes and Correctives*, Rept. RECOCP 69-2, U.S. Bureau of Reclamation, Denver, CO (1969).]

can actually develop and the plaster set can easily be disrupted by further mixing. The formation of the hemihydrate is minimized by limiting the clinker temperature prior to the initiation of grinding and by cooling the grinding mills. A small amount of hemihydrate may still be formed, but will mostly rehydrate while mixing is continuing and thus is harmless. However, a set-retarding admixture can also delay rehydration of the hemihydrate until after the mixing sequence is completed. In this way, a cement that normally behaves satisfactorily may exhibit false set.

False set may also be caused by the excessive formation of ettringite soon after mixing is completed. Ettringite has a crystal morphology similar to gypsum and thus can cause "plaster set" in an analogous way. Ettringite crystallizes most rapidly during the mixing period, but if its formation is delayed or extended in the presence of an admixture, false set can also occur. There is evidence (Chapter 8) that set-retarding admixtures can accelerate the initial hydration of C_3A and thus cause false set by

increasing the formation of ettringite. In some high-alkali cements, the formation of syngenite ($KC\overline{S}_2H$) may cause false set.

It has been found that a concrete may sometimes show abnormal set even when the crystallization of gypsum, ettringite, or syngenite cannot be implicated. It is thought that an abnormally high concentration of surface charges may cause flocculation of the paste and a high degree of thixotropy. This behavior has been called *thixotropic set*.

Flash Set If the C_3A in the cement is very reactive, flash set (or quick set) may occur. Flash set is caused by the formation of large quantities of monosulfoaluminate or other calcium aluminate hydrates. This is a rapid set that cannot be disrupted by further mixing, indicating that some strength has developed. Thus, flash set is a more severe condition than false set. Fortunately flash set has been largely eliminated as a problem with normal portland cement by the use of gypsum to control C_3A hydration. But occasionally the use of an admixture may increase the hydration of C_3A to the point at which flash set may occur. When C_3A and gypsum contents are high, the formation of ettringite can cause flash set.

Prevention of Abnormal Set Correcting a problem of abnormal set may be most simply accomplished by merely changing to another equivalent admixture, eliminating the use of the admixture, or changing the amount of gypsum added to the cement. Research has shown that a little prehydration of cement (i.e., principally C_3A) before the admixture is added may solve the problem. This could be most easily done by delayed addition of the admixture.

9.4 TESTS OF FRESH CONCRETE

Tests on fresh concrete are carried out largely as a quality-control measure and to help ensure that proper mix proportions (including admixtures) are being maintained. The underlying assumption is that tests on fresh concrete are useful for two reasons:

1. They permit some estimation of the subsequent behavior of the hardened concrete.
2. Changes in the properties of fresh concrete imply that the concrete mix is changing, so that remedial action can be taken if necessary.

By far the most common tests on fresh concrete are slump tests (or some other workability tests), as described. We will here consider the other properties of fresh concrete that may be determined.

Sampling Fresh Concrete

Concrete is a manufactured material, made from cement, aggregate, water, and admixtures. All of these ingredients are subject to variation, both in quantity and quality. Therefore, the first consideration in testing fresh concrete is that the concrete tested be *truly representative* of the batch. This is, of course, very much a matter of judgment and experience; it is difficult to lay down hard and fast rules to define "representative." Also, samples must be taken reasonably often; a common requirement is that at least one sample must be taken for every 115 m^3 (150 yd^3) of concrete cast.

A typical specification for sampling fresh concrete is the one described in ASTM C 172. This covers sampling from stationary and truck mixers and from equipment used to transport central-mixed concrete. The specification requires that tests be done on *composite* samples, that is, samples taken at several points during the discharge of the concrete, and then combined. The main problem in collecting these samples is ensuring that they are not contaminated by the forms or subgrade and that no segregation occurs during sampling. It is also important to carry out the sampling as quickly as possible (within 15 min), as workability, in particular, quickly changes with time. A minimum sample size of 0.03 m^3 (1 ft^3) is required for strength tests, but smaller samples are permitted for air content and slump tests.

It is important not only to maintain reasonable uniformity between different batches of concrete, but also to ensure that the variability of concrete from different parts of the same batch ("within-batch" variation) is low. This becomes significant when using older or worn equipment or where there is inadequate equipment maintenance. There are stringent requirements for the permissible variation of ready-mixed concrete in ASTM C 94. Samples are taken from two locations in the batch, representing the first and last portions of the batch. These samples are tested separately and the uniformity requirements are given in Table 9.1.

It is, of course, normal to expect some variability in concrete tests and quality, as will be discussed in detail in Chapter 15. However, it may be stated here as a general rule that the less variability there is in concrete properties, the better.

Time of Setting

Although the time of setting of concrete is an arbitrary measurement, it is an important parameter for a number of reasons. It is used

1. To help regulate the times of mixing and transit.
2. To gauge the effectiveness of various set-controlling admixtures (either retarding or accelerating).
3. To help plan the scheduling of finishing operations.

As we have seen, there are two tests in use for the setting time of cement, but unfortunately these do not correlate with the setting times of concrete. This is the reason for the current test procedure outlined in ASTM C 403. This penetration test is limited to concretes with slumps greater than zero and to cases "when tests of the mortar fraction of the concrete will provide the information required."

Briefly, the test consists of removing the mortar fraction of the concrete by passing it through a 4.75-mm (No. 4) sieve, rodding the mortar into containers, and then measuring the force required to cause a needle to penetrate 25.4 mm (1 in.) into the mortar. Removable needles with bearing areas ranging from 645 to 16 mm^2 (1 to 0.025 $in.^2$) are used. This is done at regular intervals, and a curve of penetration resistance versus time is plotted. Times of initial set and final set are defined as the times at which the penetration resistances are 3.5 MPa (500 lb/$in.^2$) and 27.6 MPa (4000 lb/$in.^2$), respectively. Although these are arbitrarily chosen points, 3.5 MPa corresponds approximately to the point at which the concrete will no longer become plastic under vibration; 27.6 MPa corresponds to a concrete strength of about 700 kPa (100 lb/$in.^2$).

TABLE 9.1 Requirements for Uniformity of Concrete (ASTM C 94).[a]

Test	Requirement, Expressed at Maximum Permissible Difference in Results of Tests of Samples Taken from Two Locations in the Concrete Batch
Weight per cubic foot (weight per cubic meter) calculated to an air-free basis, lb/ft^3 (kg/m^3)	1.0 (16)
Air content, vol. % of concrete	1.0
Slump	
If average slump is 4 in. (102 mm) or less, in. (mm)	1.0 (25)
If average slump is 4 to 6 in. (102 to 152 mm), in. (mm)	1.5 (38)
Coarse aggregate content, portion by weight of each sample retained on No. 4 (4.75-mm) sieve, %	6.0
Unit weight of air-free mortar[b] based on average for all comparative samples tested, %	1.6
Average compressive strength at seven days for each sample,[c] based on average strength of all comparative test specimens, %	7.5[d]

[a]Copyright ASTM INTERNATIONAL. Reprinted with permission.
[b]"Test for Variability of Constituents in Concrete," Designation 26, *Concrete Manual*, 8th ed. U.S. Bureau of Reclamation, Denver, CO (1975).
[c]Not less than three cylinders will be molded and tested from each of the samples.
[d]Tentative approval of the mixer may be granted pending results of the seven-day compressive strength tests.

ASTM C 403 has been adopted for use with shotcrete mixtures (Chapter 11), under the designation ASTM C 1117. Here also, the resistance of shotcrete panels to penetration by standard needles is measured as a function of time.

Air Content

Air-entraining agents have already been discussed in Chapter 8, and the effects of air entrainment on durability will be considered in Chapter 18. This section deals only with the techniques available to measure the amount of air in concrete. It must be remembered that in an air-entrained concrete, there will be two types of air, entrained air and entrapped air. The test methods described next are unable to distinguish between entrained air and entrapped air; they simply measure the *total* air content. Also, the air content is expressed in terms of the volume of concrete; clearly, however, air is present only in the cement paste. It should be remembered that we have already described a test for determining the air-entraining potential of cement. Unfortunately, an air-entraining cement meeting those requirements may still not give the required air content in concrete.

There are three principal tests available to measure the total air content of concrete: the *gravimetric, volumetric*, and *pressure* methods.

Gravimetric Method The gravimetric method of determining the air content (ASTM C 138) is the oldest and simplest method. It consists, basically, of comparing the unit weight of concrete containing air with the *calculated* unit weight of air-free concrete, computed from the proportions and specific gravities of the mix components. The air content, A, is then calculated as

$$A = \frac{T - W}{T} \times 100 \tag{9.3}$$

where T is the theoretical weight of the concrete on an air-free basis and W is the unit weight of concrete. This method can also be used to determine the yield (i.e., the volume of fresh concrete produced from a known quantity of ingredients) and the cement content, as well as the unit weight. This test is not suitable for field use, since it requires an accurate knowledge of the specific gravities of the ingredients. Also, the batch proportions and moisture content of the aggregates must be accurately known. It may not be used with lightweight aggregates, since their specific gravity is very difficult to determine accurately. However, the test is suitable for laboratory work, where the other parameters may be measured accurately.

Volumetric Method The volumetric method (ASTM C 173) is based on comparing the volume of fresh concrete containing air with the volume of the same concrete after the air has been expelled by agitating the concrete under water. An apparatus similar to that shown in Figure 9.16 is used.

A sample of fresh concrete is rodded into the measuring bowl having a volume of at least 0.07 ft^3 (0.002 m^3). Excess concrete is struck off, the top section clamped in place, and water added to the zero mark. The unit is then inverted and agitated vigorously until all the air seems to have been removed from the concrete and has risen to the top of the apparatus. One cup of isopropyl alcohol is then added to dispel bubbles, and a direct reading of the air content can be made by reading the amount of liquid remaining in the neck of the apparatus (correcting, of course, for the amount of isopropyl alcohol added). The chief difficulty of this test is the large amount of physical effort required to remove the air from the concrete.

Pressure Method The pressure method (ASTM C 231) is the most common method for measuring the air content of fresh concrete in the field. The basis of the method is the measurement of the change in volume of the concrete when subjected to a given pressure. This change in volume is assumed to be caused entirely by compression of the air, and the principle of Boyle's law can then be used to calculate the air content.

An apparatus similar to that shown in Figure 9.17 is used. The concrete is consolidated into the measuring bowl and struck off level to the rim, and the apparatus is assembled. Water is added through the petcocks, and the test pressure applied and adjusted using the air bleeder valve. The pressure is released, and the air content is read. This gives the apparent air content, A_1. (Similarly, the air content of the aggregate itself, A_2, is determined for porous aggregates.) Then the true air content is simply $A_1 - A_2$. The chief advantage of this method is that it is not necessary to determine the specific gravities, moisture contents, or mix proportions of the components. If the

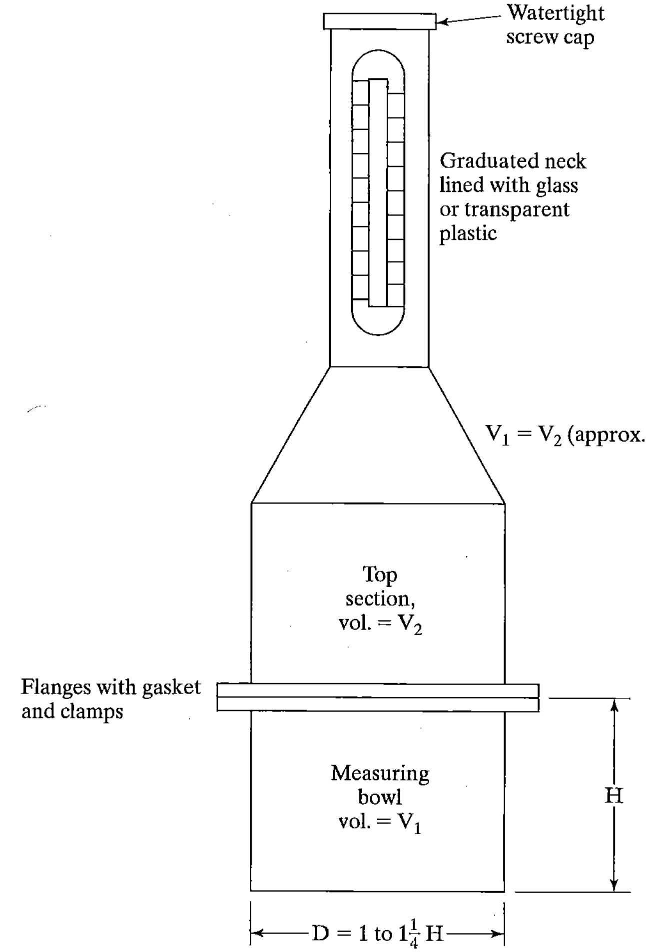


FIGURE 9.16

Apparatus for measuring air content of fresh concrete by volumetric method. (From ASTM C 173. Copyright ASTM INTERNATIONAL. Reprinted with permission.)

air content of the concrete exceeds the range of the apparatus, the determination can be repeated at a lower test pressure.

It should be noted that the foregoing tests measure only the total volume of air in the concrete. They yield no information about the nature of the air-void system. However, as we have seen in Chapter 8, it is the nature of the air-void system (i.e., the pore size and pore spacing) that determines the durability of the concrete. To determine these, it is necessary to examine the void system of the hardened concrete itself. This may be done using the procedures described in ASTM C 457 (see section 8.3).

Unit Weight and Yield

The density (unit weight) of the fresh concrete can be determined by weighing a known volume of concrete. It is usual to weigh the sample of concrete used for the air

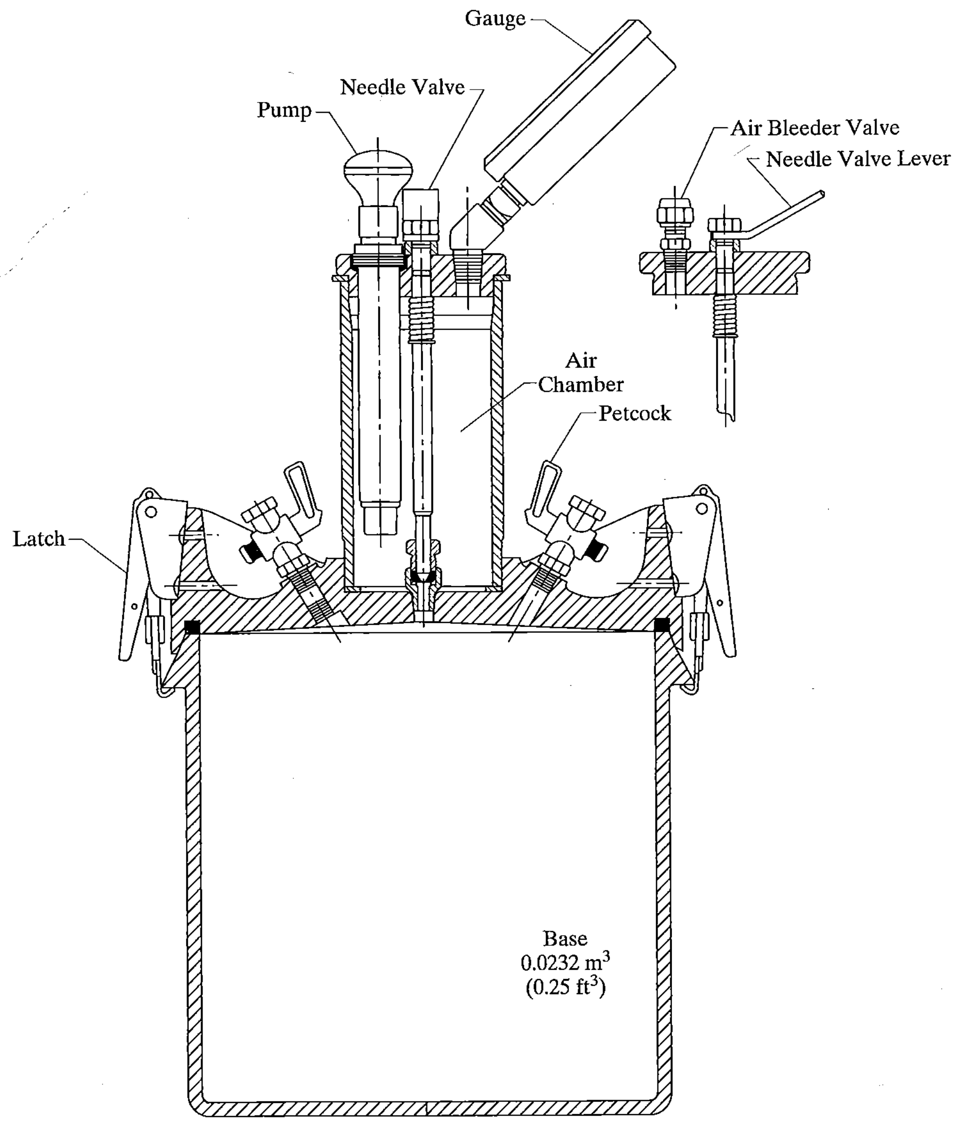


FIGURE 9.17

Pressure method for air content, Type B meter. (Forney Press-Aire Meter. Courtesy of Forney, Inc.)

measurement immediately before the air content is determined, since this is a known volume of concrete. The presence of entrained air affects the unit weight, since air contributes to volume, but not to weight. The unit weight can be used as an approximate indication of air content for concretes made with the same materials.

The unit weight can also be used to determine the volume of the concrete batch

$$V = \frac{w}{\text{unit weight}} \text{m}^3(\text{ft}^3) \quad (9.4)$$

where, w is the combined weight of the concrete constituents, including water. Thus, the *yield* of the concrete can be determined:

$$Y = \frac{V}{w_{\text{cem}}} \text{m}^3/\text{kg}(\text{ft}^3/\text{lb}) \quad (9.5)$$

This tells us how much concrete can be made using a given weight of cement.

Rapid Analysis of Fresh Concrete

The tests that we have described so far are really indirect methods of trying to ascertain that the mix proportions in the concrete being tested are really the same as those specified. Clearly, the quality-control requirements would be largely met if we could, in fact, determine the proportions of all the ingredients in the mix. Then a simple strength test at some age (say, seven days) would suffice to ensure that some unforeseen accident was not preventing the concrete from hardening properly. In fact, even a test to measure directly the two most important parameters, the cement content and the *w/c* ratio, would be very useful. However, there is currently no test method available that provides sufficient accuracy, but is still relatively easy to carry out. In addition, no reliable test is yet available to determine the type and amount of admixtures present (which is complicated further by the fact that the exact chemical composition of many admixtures is a closely guarded secret).

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PROBLEMS

9.1. Discuss the implications of the one-point workability tests currently in use.
9.2. Why is the slump test universally used in North America?
9.3. What is slump loss and what factors affect it?
9.4. Without changing the water content, what other strategies could you use to change the workability of concrete?
9.5. What are the practical consequences of bleeding?

9.6. Describe the process of setting.
9.7. Do measurements of initial and final set correspond to fundamental changes in the hydration process?
9.8. Compare false set, flash set, and plaster set.
9.9. What is the importance of the sampling procedures used to obtain fresh concrete for testing?
9.10. Would it be desirable to have an air content test that distinguishes between entrained air and entrapped air?

CHAPTER 10

Proportioning Concrete Mixes

The proportioning of concrete mixtures, more commonly referred to as *mix design*, is a process that consists of two interrelated steps: (1) selection of the suitable ingredients (cement, aggregate, water, and admixtures) of concrete and (2) determining their relative quantities ("proportioning") to produce, as economically as possible, concrete of the appropriate workability, strength, and durability. These proportions will depend on the particular ingredients used, which will themselves depend on the application. Other criteria, such as designing to minimize shrinkage and creep or for special chemical environments, may also be considered. However, although a considerable amount of work has been done on the theoretical aspects of mix design, it still remains largely an empirical procedure. And, although many concrete properties are important, most design procedures are based primarily on achieving a specified compressive strength at some given workability and age; it is assumed that if this is done, the other properties (except perhaps resistance to freezing and thawing or other durability problems, such as resistance to chemical attack) will also be satisfactory. But before turning to the mix design methods now in common use, it is worthwhile to examine the basic design considerations themselves in more detail.

10.1 BASIC CONSIDERATIONS

Economy

The cost of concrete is made up of the costs of materials, labor, and equipment. However, except for some special concretes, the costs of labor and equipment are largely independent of the type and quality of concrete produced. It is therefore the material costs that are most important in determining the relative costs of different mix designs. Since cement is much more expensive than aggregate, it is clear that minimizing the cement content is the most important single factor in reducing concrete costs. This can, in general, be done by using the lowest slump that will permit adequate placement, by using the largest practical maximum size of aggregate, by using the optimum ratio of

coarse to fine aggregates, and, where necessary, by using appropriate admixtures. It should be noted here that in addition to cost, there are other benefits to using a low cement content; shrinkage will in general be reduced, and there will be less heat of hydration. However, if the cement contents are too low, they will diminish the early strength of the concrete and will make uniformity of the concrete a more critical consideration.

The economy of a particular mix design should also be related to the degree of quality control that can be expected on a job. As will be discussed in Chapter 15, the mean concrete strength must be higher than the specified minimum compressive strength because of the inherent variability of concrete. On small jobs, it may be cheaper to "overdesign" the concrete than to implement a quality-control program in conjunction with a more cost-efficient concrete.

Workability

Clearly, a properly designed mix must be capable of being placed and compacted properly *with the equipment available*. Finishability must be adequate, and segregation and bleeding should be minimized. As a general rule, the concrete should be supplied at the *minimum* workability that will permit adequate placement. For concretes without mineral admixtures, the water requirement for workability depends mostly on the characteristics of the aggregate rather than those of the cement. Where necessary, workability should be improved by redesigning the mix to increase the mortar content rather than by simply adding more water or more fine material. Thus, cooperation between the mix designer and the contractor is essential to ensure a good concrete mix. In some cases, a less economical mix may be the best solution. A deaf ear should be turned to the frequent pleas from any job site for "more water." However, prudent planning is needed to ensure that concrete with the required workability is delivered.

Strength and Durability

In general, concrete specifications will require a minimum compressive strength. They may also impose limitations on the permissible *w/c* ratios and minimum cement contents. It is important to ensure that these requirements are not mutually incompatible. As we will see in Chapter 19, it is not necessarily the 28-day strength that is most important; strengths at other ages may control the design. Specifications may also require that the concrete meet certain durability requirements, such as resistance to freezing and thawing or chemical attack. These considerations may provide further limitations on the *w/c* ratio or cement content and in addition may require the use of admixtures. The process of mix design, then, involves the satisfactory resolution of all the requirements just described. Since these requirements cannot all be optimized simultaneously, some compromises (as between strength and workability) will be necessary. It must be remembered that even a "perfect" mix will not perform properly unless the proper placing, finishing, and curing procedures are carried out.

10.2 FUNDAMENTALS OF MIX DESIGN

There have been two aspects of mix design in which most of the theoretical work has been carried out: water content and aggregate grading. Most of the modern empirical design methods depend heavily on these two considerations.

Water/Cement Ratio

In 1918, Duff Abrams enunciated his water/cement ratio law for the strength of concrete: "For given materials, the strength depends only on one factor—the ratio of water to cement."[1] This can be expressed by a formula in the form

$$\sigma_c = \frac{A}{B^{1.5(w/c)}} \tag{10.1}$$

where σ_c = compressive strength at some fixed age, A = empirical constant, B = constant that depends mostly on the cement properties, and w/c = water/cement ratio by weight. Usually, A is taken to be 96.5 MPa (14,000 lb/in.^{2}), and B depends on the type of cement, but may be taken to be about 4. This observation, that strength is inversely proportional to the *w/c* ratio, remains the basis for most mix design procedures. The reason for this was shown by Powers and Brownyard[2] to be the fact that the *w/c* ratio determines the porosity of the cement paste. This was discussed in terms of the gel/space ratio concept in Chapter 4. Of course, Eq. (10.1) must be normalized in terms of the maturity (degree of hydration) of the cement. It might be noted as well that, since the *w/c* ratio controls the porosity, it also largely controls the durability of the concrete, as will be discussed in Chapter 18.

Aggregate Grading

Grading of granular materials to produce dense packing is of broad industrial interest. An "ideal" curve determined by Fuller and Thompson in 1907 is given by

$$P_t = \left(\frac{d}{D}\right)^{1/2} \tag{10.2}$$

where P_t = fraction of total solids finer than size d and
D = maximum particle size.

This "Fuller curve" is a special case of a more general equation developed by Andreasson.

$$P_t = \left(\frac{d}{D}\right)^{q} \tag{10.3}$$

where q lies between 0 and 1.

It may be shown that when *all* aggregate sizes below D are present, the void content depends only on q, decreasing as q decreases. Although values of q between $\frac{1}{3}$ and $\frac{1}{2}$ have been used for concrete, the lowest practical value of q is about $\frac{1}{2}$, the value recommended by Fuller and Thompson. The reason is that minimizing void content, to give more economical concrete that has a minimum cement, results in concrete with

[1]Feret in 1896 had formulated a more general "law," expressing concrete strength in terms of the cement/total voids ratio, of which Abrams's law is a special case.

[2]T. C. Powers and T. L. Brownyard, "Studies of the Physical Properties of Hardened Portland Cement Paste," Bulletin 22, Research Laboratories of the Portland Cement Association (1948).

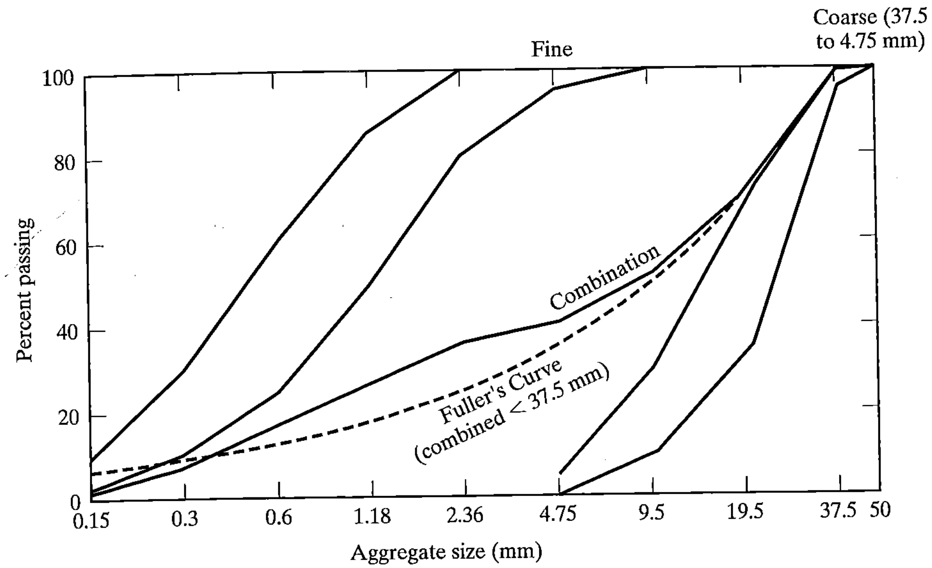


FIGURE 10.1

Grading curves indicating the specified grading limits (ASTM C 33) for fine aggregate and one size of coarse aggregate. A combined grading curve (40% fine aggregate and 60% coarse aggregate for the midrange of the ASTM limits) and the corresponding Fuller–Thompson "ideal" grading curve are also shown.

poor workability. In addition, very fine particles cannot form densely packed aggregates; packing density decreases as the average particle size decreases. To improve concrete behavior, ASTM C 33 (and CSA A23.1) requires that 2 to 10% of the fine aggregate must pass the 150-μm (No. 100) sieve, and 10 to 30% must pass the 300-μm (No. 50) sieve. In terms of packing, the gradations used in ASTM C 33 can only approximate the Fuller curve, as shown in Figure 10.1.

Shilstone[3] has developed an empirical procedure that uses a combined aggregate gradation to optimize concrete workability, economy, and strength. As shown in Figure 10.2, the combined aggregate is characterized as consisting of material equal to and larger than 9.5 mm ($^3/_8$ in.) (designated as Q for *quality* filler), material smaller than 9.5 mm ($^3/_8$ in.) and retained on the 2.36-mm (No. 8) sieve (designated as I for *intermediate* particles that fill major voids and aids in mix "mobility"), and material smaller than 2.36-mm (designated as W for particles that aid *workability*). Shilstone observed that experimentally "optimum" combined aggregate gradations can be expressed by the relationship between a *workability factor* [the fraction of total aggregate passing the 2.36-mm (No. 8) sieve = $W/(Q + I + W)$] and a *coarseness factor* [ratio between the weight fraction of particles retained on the 9.5-mm ($^3/_8$-in.) sieve to the weight of all particles retained on the 2.36-mm (No. 8) sieve = $Q/(Q + I)$]. Mixes

[3]J. M. Shilstone, Sr., "Concrete Mixture Optimization," *Concrete International*, Vol. 12, No. 6, pp. 33–39 (June 1990).

within the band shown in Figure 10.3 are expected to produce cohesive concrete that can be readily consolidated by high-frequency vibration and that will generally exhibit

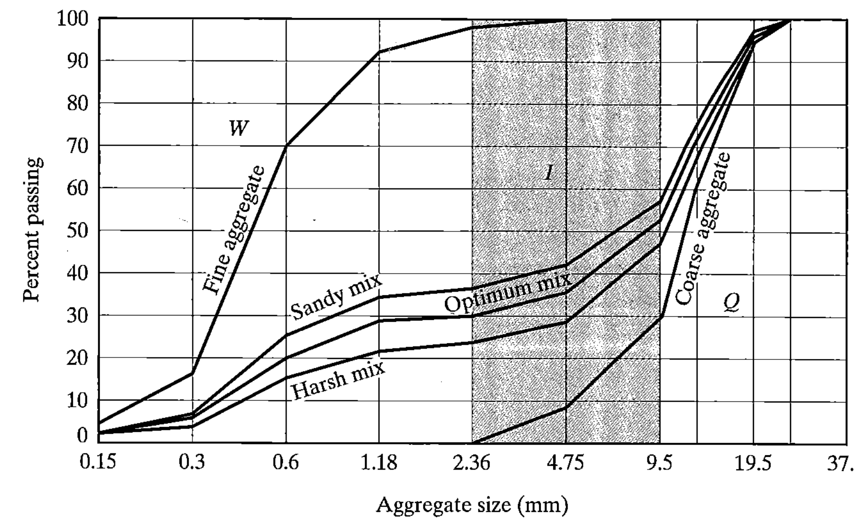


FIGURE 10.2

Combined aggregate gradations, including designations of size fractions as particles that aid workability (W), intermediate particles (I), and quality filler (Q). (Adapted from J. M. Shilstone, Sr.)

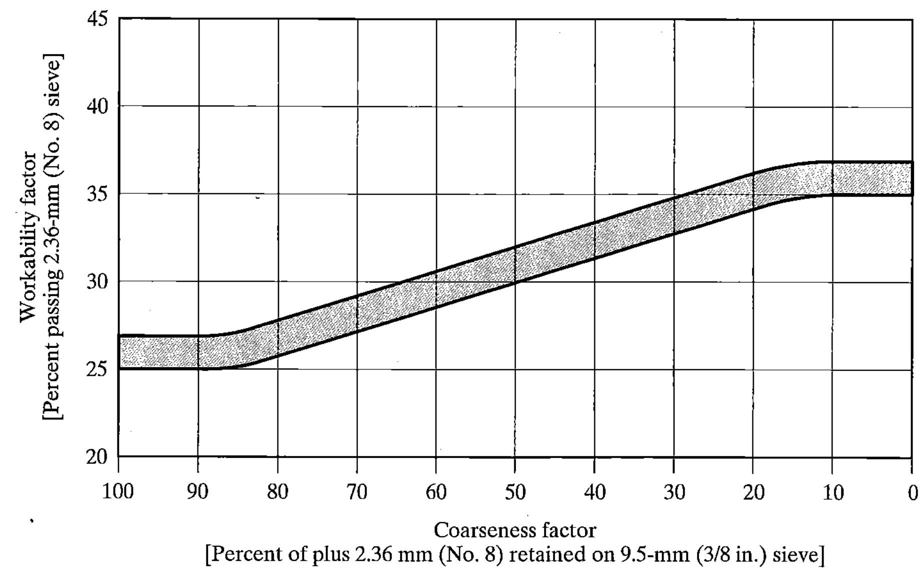


FIGURE 10.3

Band of aggregate gradations on coarseness factor chart that produce optimum concrete mixtures. (Adapted from J. M. Shilstone, Sr.)

the lowest water demand for a given cement content. The concrete used to develop Figure 10.3 contained 335 kg/m^3 (564 lb/yd^3) of cement. Since cement particles also aid workability, the *W* factor should be adjusted by treating any increase or decrease from this cement content as an increase or decrease in the *W* content of the combined aggregate (accounting for differences in the specific gravities of the fine aggregate and cement).

The grading limits for fine and coarse aggregate shown in Figure 10.1 are, like the combined gradations in Figures 10.2 and 10.3, based on practical experience rather than on theory. In fact, it is possible to make a satisfactory concrete from almost any type of aggregate grading, although gradations outside the limits of Figure 10.1 (and even some gradations within these limits) may be uneconomical and difficult to handle with regard to segregation, consolidation, and finishing. The procedures developed by Shilstone are fairly new and are not, as yet, standard practice in the concrete industry.

10.3 ACI METHOD OF MIX DESIGN

There are a number of different methods of mix design available. Although they are not directly comparable, they do give approximately the same relative proportions of materials, and all are capable of yielding suitable concrete mixes. The most common method used in North America is that established by ACI Standard Practice 211.1, and this method will be described here in some detail. It must be remembered, however, that any mix design method will provide only a first approximation of proportions. These must be checked by trial batches in the laboratory or in the field and then adjusted as necessary to produce the desired concrete characteristics. With any given set of materials, it may be found that considerable deviations from the ACI recommended practice may be necessary. Once sufficient experience with local materials is acquired, the tables used in the ACI method should be modified to take their properties into account.

As was stated earlier, the job specifications may dictate certain mix requirements, such as minimum cement contents and *w/c* ratios, slump, air content, maximum aggregate size, strength, the use of admixtures, or other special requirements. But regardless of the specification requirements, the establishment of the batch weights [note that it is nearly always preferable to batch concrete by *weight* (Chapter 11) rather than by volume] can best be accomplished by following the sequence of steps laid out next. This will ensure that the characteristics of the available materials are properly considered in combining them into a suitable concrete mixture. In summary, the mix design process consists of (1) determining the job parameters—aggregate properties, maximum aggregate size, slump, *w/c* ratio, and admixtures; (2) calculating the batch weights; and (3) adjusting to the batch weights based on a trial mix made according to these calculations.

Mix Design Procedures

1. **Required information.** Before starting the mix design process, information is required on both the materials to be used and the structure into which the concrete will be placed. Raw material properties include sieve analyses of both the fine and coarse aggregates, unit weight of the coarse aggregate, bulk specific gravities, and absorption capacities of the aggregates. Information on the structure includes the type and dimensions of the structural members, the minimum space between reinforcing bars, the required concrete strength, and the exposure conditions to which the concrete will be subjected.

2. **Choice of slump.** Usually, slump will be specified for a particular job, to take into account the anticipated methods of handling and placing the concrete. However, where the slump has not been specified, appropriate values can be chosen from Table 10.1, which applies when the concrete is to be consolidated by vibration. As a general rule, the lowest slump that will permit adequate placement should be selected. This, however, does not mean that the minimum slump should automatically be chosen from Table 10.1. "Adequate placement" requires the consideration of job-site conditions, which often leads to the selection of a slump closer to the maximum recommended value. It is becoming increasingly common to design a concrete for a lower slump and then increase it to a higher value using a water-reducing admixture.

3. **Maximum aggregate size.** Generally, the largest maximum size of aggregate available (and consistent with the limitations to be listed) should be used, as this will minimize the required cement content. The limitations on maximum aggregate size (see Section 7.1) are as follows:

   a. For reinforced (or prestressed) concrete, the maximum size may not exceed one-fifth of the minimum dimension between forms, or three-fourths of the minimum

TABLE 10.1 Recommended Slumps for Various Types of Construction.[a,b]

	Slump (mm)		*Slump (in.)*	
Types of Construction	*Max.*[c]	*Min.*	*Max.*[c]	*Min.*
Reinforced foundation walls and footings	75	25	3	1
Plain footings, caissons, and substructure walls	75	25	3	1
Beams and reinforced walls	100	25	4	1
Building columns	100	25	4	1
Pavements and slabs	75	25	3	1
Mass concrete	50	25	2	1

[a]From ACI 211.1. Reproduced with permission.
[b]Slump may be increased when chemical admixtures are used, provided that the admixture-treated concrete has the same or lower water–cement ratio or water–cementitious material ratio and does not exhibit segregation potential or excessive bleeding.
[c]May be increased 25 mm (1 in.) for methods of consolidation other than vibration.

clear spacing between bars, strands, bundles of bars, or between the steel and the formwork.[4]

b. For slabs, the maximum size may not exceed one-third the slab depth.

If it is shown by experience that it is possible to place the concrete without honeycombing or voids, these requirements may be relaxed. It has also been found that, at a given *w/c* ratio, higher compressive strengths can be achieved with smaller maximum sizes of aggregates. There is, thus, a trend toward the use of reduced maximum aggregate sizes, especially for high-strength concretes (see Chapter 19). In many areas, the largest sizes available are 19 or 25 mm ($^3\!/_4$ in. or 1 in.). It should also be remembered that, for a given job, it may be unwise to recommend different maximum aggregate sizes for different parts of the structure, as this may lead to confusion and increases the probability of error.

4. **Estimation of mixing water and air content.** As we have seen in Chapter 9, the workability of concrete is dependent primarily on the paste content of the concrete; the amount of entrained air; and the maximum size, grading, and particle shape of the aggregate. An estimate of the water requirement to produce different slumps for both air-entrained and non-air-entrained concrete can be obtained from Table 10.2, which is based on experience obtained over many years of practice. It is, however, better to establish these numbers from experience with the actual materials in question rather than resorting to the use of Table 10.2. When water-reducing admixtures are used, these values should be decreased according to the amount of water reduction anticipated. Table 10.2 also shows the approximate amount of entrapped air to be expected in non-air-entrained concrete and gives the recommended levels of air entrainment (when required) for different maximum sizes of aggregate and for three different levels of severity of exposure. Since there is frequent pressure from the job site for concrete with "more slump" (which is often obtained by adding more water rather than by adding more paste, leading to a higher *w/c* ratio and lower strength), it is suggested that the trial batches of the mix used to develop the strength relationships be made to represent the most unfavorable combination of air content and water content. That is, both the maximum allowable air content and the maximum allowable slump should be used in these batches rather than using average values. This will help prevent overestimating the strength.

5. **Water/cement or water/cementitious material ratio.** The selection of the appropriate *w/c* or *w/cm* ratio may be governed not only by strength, but also by durability requirements:

a. *Strength.* In the absence of strength vs. *w/c* ratio data for the specific materials, a conservative estimate can be made for the expected 28-day compressive strength from Table 10.3, when Type I portland cement is used, either alone or together

[4]ACI 211.1 and ACI 318 (*Building Code Requirements*) do not impose the limitation based on the spacing between steel and formwork. This oversight can result in voids and honeycombing as larger aggregate particles become wedged between the reinforcing steel and the face of the form.

TABLE 10.2 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregates.[a,b]

Slump		Water, kg/m³ (lb/yd³) of Concrete for Indicated Nominal Maximum Sizes of Aggregate													
mm	in.	9.5 mm	(3/8 in.)	12.5 mm	(1/2 in.)	19 mm	(3/4 in.)	25 mm	(1 in.)	37.5 mm	(1½ in.)	50[c] mm	(2[c] in.)	75[c] mm	(3[c] in.)
Non-Air-Entrained Concrete															
30–50	1–2	210	(350)	200	(335)	185	(315)	180	(300)	160	(275)	155	(260)	130	(220)
80–100	3–4	225	(385)	215	(365)	200	(340)	195	(325)	175	(300)	170	(285)	145	(245)
150–180	6–7	240	(410)	230	(385)	210	(360)	205	(340)	185	(315)	180	(300)	160	(270)
Approximate amount of air in non-air-entrained concrete, %		3		2.5		2		1.5		1		0.5		0.3	
Air-Entrained Concrete															
30–50	1–2	180	(305)	175	(295)	165	(280)	160	(270)	145	(250)	140	(240)	120	(205)
80–100	3–4	200	(340)	190	(325)	180	(305)	175	(295)	160	(275)	155	(265)	135	(225)
150–180	6–7	215	(365)	205	(345)	190	(325)	185	(310)	170	(290)	165	(280)	155	(260)
Recommended average total air content, %, for level of exposure:															
Mild exposure		4.5		4.0		3.5		3.0		2.5		2.0		1.5[d]	
Moderate exposure		6.0		5.5		5.0		4.5		4.5		4.0		3.5[d]	
Severe exposure[e]		7.5		7.0		6.0		6.0		5.5		5.0		4.5[d]	

[a]Adapted from ACI 211.1. Reproduced with permission.

[b]These quantities of mixing water are for use in computing cement factors for trial batches. They are maxima for reasonably well-shaped, angular, coarse aggregates graded within limits of accepted specifications.

[c]The slump values for concrete containing aggregate larger than 37.5 mm (1½ in.) are based on slump tests made after removal of particles larger than 37.5 mm (1½ in.) by wet screening.

[d]For concrete containing large aggregates, which will be wet screened over the 37.5-mm (1½-in.) sieve prior to testing for air content, the percentage of air expected in the 37.5 mm (1½ in.) minus material should be as tabulated in the 37.5-mm (1½ -in.) column. However, initial proportioning calculations should include the air content as a percent of the whole.

[e]These values are based on the criterion that 9% air is needed in the mortar phase of the concrete. If the mortar volume will be substantially different from that determined in this recommended practice, it may be desirable to calculate the needed air content by taking 9% of the actual mortar volume.

with a pozzolan.[5] Of course, it is possible that the specifications may be based on a required strength at a time other than 28 days (e.g., at the time when forms are to be stripped), or the design may require the use of another standard type of portland cement (Types II, III, IV, or V), blast furnace slag, or very high quantities of pozzolans. In these cases, Table 10.3 is not applicable. The designer must develop his or her own data for these cases, or when the design is governed by a flexural strength requirement. It is always more desirable to develop the appropriate strength-time–*w/cm* ratio relationships for the materials that are actually to be used on the job. In this way, the effects of admixtures can also be determined.

[5]Although ACI 211.1 does not specifically address the use of blast furnace slag, the procedures described here can be applied equally well for all mineral admixtures.

TABLE 10.3 Relationships between Water/Cement or Water/Cementitious Materials Ratio and Compressive Strength of Concrete.[a]

Compressive Strength at 28 days[b]	Water/Cement Ratio, by Weight: Non-Air-Entrained Concrete	Air-Entrained Concrete
45 MPa	0.37	—
40 MPa	0.42	—
35 MPa	0.47	0.39
30 MPa	0.54	0.45
25 MPa	0.61	0.52
20 MPa	0.69	0.60
15 MPa	0.79	0.70
6000 lb/in.^{2}	0.41	—
5000 lb/in.^{2}	0.48	0.40
4000 lb/in.^{2}	0.57	0.48
3000 lb/in.^{2}	0.68	0.59
2000 lb/in.^{2}	0.82	0.74

[a]Adapted from ACI 211.1. Reproduced with permission.
[b]Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table 10.2. Strength is based on 150 × 300 mm (6 × 12 in.) cylinders moist-cured in accordance with ASTM C 31.

b. *Durability*. If there are severe exposure conditions, such as freezing and thawing or exposure to seawater or sulfates, the more severe *w/c* ratio requirements of Table 10.4 may govern. It might be noted that other standards may have somewhat different requirements. For instance, CSA Standard A23.1 provides a more detailed guide to the selection of maximum permissible *w/c* ratios, concrete strengths, and air contents for severe exposure conditions. These requirements are shown in Tables 10.5 and 10.6. Table 10.7 shows the ACI requirements for concretes exposed to different concentrations of sulfates in soils or groundwaters, which include a specification of the type of cement to be used (see Chapter 18).

6. **Calculation of cement or cementitious material content.** Once the water content and *w/c* ratio (Steps 4 and 5) are determined, the amount of cement or cement plus mineral admixture per unit volume of concrete is determined simply by dividing the estimated water requirement by the *w/c* ratio. However, many specifications, in addition, require a *minimum cement content*. Such a requirement may be used to ensure satisfactory finishability, quality of vertical surfaces, or workability; it may also ensure against low strengths due to increased water demands at the job site.

When a portion of the cement is replaced by a mineral admixture, the replacement can be made on the basis of mass (weight) or volume, although mass is most commonly used. Since portland cement and the admixture have different specific gravities, the values of the weight and volume percentages will differ.

TABLE 10.4 Maximum Permissible Water/Cement or Water/Cementitious Material Ratios for Concrete in Severe Exposures.[a,b]

Type of Structure	Structure Wet Continuously or Frequently, and Exposed to Freezing and Thawing[c]	Structure Exposed to Seawater or Sulfates
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 25-mm (1-in.) cover over steel	0.45	0.40[d]
All other sections	0.50	0.45[d]

[a]Adapted from ACI 211.1. Reproduced with permission.
[b]Based on report of ACI Committee 201, Durability of Concrete in Service. Cementitious materials other than cement should conform to ASTM C 618 and C 989.
[c]Concrete should also be air entrained.
[d]If sulfate-resisting cement (Type II or Type V of ASTM C 150) is used, permissible water/cement or water/cementitious material ratio may be increased by 0.05.

The following relationships allow for conversion between weight fraction, F_w, and volume fraction, F_v:

$$F_w = \frac{1}{1 + \left(\frac{G_c}{G_p}\right)\left(\frac{1}{F_v} - 1\right)} \quad (10.4)$$

$$F_v = \frac{1}{1 + \left(\frac{G_p}{G_c}\right)\left(\frac{1}{F_w} - 1\right)} \quad (10.5)$$

where F_w and F_v = ratio of mineral admixture weight or volume, respectively, to weight or volume of total cementitious material; G_c = specific gravity of cement (generally 3.15); G_p = specific gravity of mineral admixture.

When calculating the cementitious material content, it is easiest to first calculate the total weight of cementitious material on the basis of 100% cement, C_T, and then to calculate the final weights of the individual cementitious materials based on the desired weight or volume replacement percentage. On a weight replacement basis, the weight of pozzolan is $P = F_w C_T$, and the weight of cement is $C = (1 - F_w)C_T$. On a volume replacement basis, $P = F_v(G_p/G_c)C_T$ and $C = (1 - F_v)C_T$. All common mineral admixtures have specific gravities lower than portland cement, so that when replaced on a mass basis, they occupy a larger volume than the cement they replace.

7. **Estimation of coarse aggregate content.** It has been found empirically that aggregates having the same maximum size and grading will yield workable mixes when used in concrete in the volumes (on a dry-rodded basis)[6] shown in Table 10.8. For the same workability, the volume of coarse aggregate depends only on its maximum size and on

[6]Oven-dry rather than air-dry.

TABLE 10.5 Minimum Specified Compressive Strengths and Maximum Permissible Water/Cementitious Material Ratios for Different Types of Structures and Varying Degrees of Exposure.[a]

Class of Exposure[b]	*Condition of Exposure*	*Minimum Specified 28-day Compressive Strength, MPa (lb/in.²)*	*Maximum Water/Cementitious Material Ratio*	*Air Content Category*
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of marine structures located within the tidal zone.	35 (5100)	0.40	[c]
		32 (4600)	0.45	1
C-2	Non-structurally reinforced (i.e., plain) concrete exposed to chlorides and freezing and thawing. Examples: pavements, sidewalks, curbs and gutters.			
C-3	Continuously submerged concrete exposed to chlorides, but not freezing and thawing. Examples: underwater portions of marine structures.	30 (4400)	0.50	2
C-4	Non-structurally reinforced concrete exposed to chlorides, but not freezing and thawing. Examples: interior parking slabs-on-grade.	25 (3600)	0.55	2
F-1	Concrete exposed to freezing and thawing in a saturated condition, but not chlorides. Examples: exterior walls and columns.	30 (4400)	0.50	1
F-2	Concrete in an unsaturated condition exposed to freezing and thawing. Examples: exterior walls and columns.	25 (3600)	0.55	2
N	Concrete exposed to neither chlorides nor freezing and thawing. Examples: interior slabs, walls and columns.	for structural design	for structural design	

[a] Adapted from CSA Standard A23.1, *Concrete Materials and Methods of Concrete Construction*, which is copyrighted by the Canadian Standards Association. Copies may be purchased from the Association, 178 Rexdale Boulevard, Rexdale, Ontario M9W 1R3.
[b] "C" classes pertain to chloride exposure.
"F" classes pertain to freezing and thawing exposure only.
"N" class pertains to nonexposure to either chlorides or freezing and thawing.
[c] Use Category 1 for concrete exposed to freezing and thawing. Use Category 2 for concrete not exposed to freezing and thawing (see Table 10.6).

TABLE 10.6 Range of Total Air Content for Various Classes of Exposure as Defined in Table 10.5.[a]

Air Content Category	*Range in Air Content for Concrete with Indicated Nominal Maximum Sizes of Coarse Aggregate, %*		
	10 mm	*14–20 mm*	*28–40 mm*
1	6 to 9	5 to 8	4 to 7
2	5 to 8	4 to 7	3 to 6

[a] Adapted from CSA Standard A23.1. See note to Table 10.5.

TABLE 10.7 Requirements for Concrete Exposed to Sulfate-Containing Solutions.[a]

Sulfate Exposure	*Water-Soluble Sulfate (SO_4) in Soil, Percent By Mass*	*Sulfate (SO_4) in Water, ppm*	*Cement Type*	*Maximum Water–Cementitious Material Ratio, by Mass, Normal Weight Aggregate Concrete*[b]	*Minimum Specified Compressive Strength Normal Weight and Lightweight Aggregate Concrete, MPa*[b] *(lb/in.²)*
Negligible	$0.00 \le SO_4 < 0.10$	$0 \le SO_4 < 150$	–	–	–
Moderate[c]	$0.10 \le SO_4 < 0.20$	$150 \le SO_4 < 1500$	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	28 (4000)
Severe	$0.20 \le SO_4 \le 2.00$	$1500 \le SO_4 \le 10{,}000$	V	0.45	31 (4500)
Very severe	$SO_4 > 2.00$	$SO_4 > 10{,}000$	V plus pozzolan[d]	0.45	31 (4500)

[a] Adapted from ACI 318. Reproduced with permission.
[b] A lower water–cementitious material ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing.
[c] Seawater.
[d] Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

TABLE 10.8 Volume of Coarse Aggregate per Unit Volume of Concrete.[a]

Maximum Size of Aggregate		*Volume of Dry-Rodded Coarse Aggregate*[b] *per Unit Volume of Concrete for Different Fineness Moduli of Sand*			
mm	*in.*	2.40	2.60	2.80	3.00
9.5	$\frac{3}{8}$	0.50	0.48	0.46	0.44
12.5	$\frac{1}{2}$	0.59	0.57	0.55	0.53
19	$\frac{3}{4}$	0.66	0.64	0.62	0.60
25	1	0.71	0.69	0.67	0.65
37.5	$1\frac{1}{2}$	0.76	0.74	0.72	0.70
50	2	0.78	0.76	0.74	0.72
75	3	0.82	0.80	0.78	0.76
150	6	0.87	0.85	0.83	0.81

[a] Adapted from ACI 211.1. Reproduced with permission.
[b] Volumes are based on aggregates in dry-rodded condition as described in ASTM C 29. For less workable concrete, such as that required for concrete-pavement construction, they may be increased about 10%. For more workable concrete, such as sometimes may be required when concrete must be placed around congested reinforcing steel or when placement is to be by pumping, they may be reduced up to 10%.

the fineness modulus of the fine aggregate. The OD weight of coarse aggregate required per cubic meter of concrete is simply equal to the value from Table 10.8 multiplied by the dry-rodded unit weight of the aggregate in kg/m³. (The volume of aggregate in ft³/yd³ of concrete can be obtained by multiplying the appropriate value by 27. This volume can then be converted to an OD weight by multiplying it by the dry-rodded weight per cubic foot of coarse aggregate.) To convert from OD to SSD weights, multiply by $(1 + A/100)$ as in Eq. (7.2) (Section 7.1).

TABLE 10.9 First Estimate of Weight of Fresh Concrete.[a]

Maximum Size of Aggregate		First Estimate of Concrete Weight[b]			
		Non-Air-Entrained Concrete		Air-Entrained Concrete	
mm	*in.*	*kg/m³*	*lb/yd³*	*kg/m³*	*lb/yd³*
9.5	$\frac{3}{8}$	2285	3840	2200	3710
12.5	$\frac{1}{2}$	2315	3890	2230	3760
19	$\frac{3}{4}$	2355	3960	2280	3840
25	1	2375	4010	2285	3850
37.5	$1\frac{1}{2}$	2420	4070	2320	3910
50	2	2445	4120	2345	3950
75	3	2490	4200	2400	4040

[a]Adapted from ACI 211.1. Reproduced with permission.
[b]Values calculated by Eqs. (10.6) and (10.7) for concrete containing 330 kg/m³ (550 lb/yd³) of cement, slump of 75 to 100 mm (3 to 4 in.), and aggregate bulk specific gravity of 2.7.

8. **Estimation of fine aggregate content.** We have now established the weights of all of the concrete ingredients except that of the fine aggregate. We can establish fine aggregate content in two ways:

a. *"Mass" ("Weight") Method.* This requires a knowledge of the weight (per m³ or yd³) of the fresh concrete, which can often be estimated from previous experience with the materials in question; failing this, Table 10.9 may be used as a first estimate.

An exact calculation of the weight of fresh concrete in kg/m³ or lb/yd³ can be obtained as follows:

$$U_m = 10G_a(100 - \bar{A}) + C_m\left(1 - \frac{G_a}{G_c}\right) - W_m(G_a - 1) \quad \text{kg/m}^3 \tag{10.6}$$

$$U = 16.85G_a(100 - \bar{A}) + C\left(1 - \frac{G_a}{G_c}\right) - W(G_a - 1) \quad \text{lb/yd}^3 \tag{10.7}$$

where $U_m, (U)$ = weight of fresh concrete, kg/m³(lb/yd³); G_a = weighted average bulk specific gravity (SSD)[7] of combined fine and coarse aggregate, assuming reasonable weight proportions; G_c = specific gravity of cement (generally 3.15); $\bar{A}$ = air content, %; $W_m\,(W)$ = mixing water content, kg/m³ (lb/yd³); and $C_m\,(C)$ = cement requirement, kg/m³ (lb/yd³).

If the first estimate of the weight of the fresh concrete is not very good, an iterative procedure may be required to obtain G_a. The weight of fine aggregate is then the difference between the total weight of the fresh concrete and the weight of the other ingredients.

[7]If the aggregate is in a condition other than SSD, the appropriate BSG should be used.

b. *"Volume" Method.* This is the preferred method, as it is a somewhat more exact procedure, which requires a knowledge of the volumes displaced by the various ingredients. That is, the volumes of the cementitious material, water, air, and coarse aggregate are subtracted from the total volume; the difference is the volume of fine aggregate. The weight of fine aggregate can then be obtained by multiplying this volume by the density of the fine aggregate.

The aggregate calculations just described are best carried out using SSD weights, but they can be done by using OD weights. In this case, the coarse aggregate weight is converted to OD by measuring its absorption capacity and then using the OD bulk specific gravity.

9. **Adjustment for moisture in the aggregates.** The actual water content of the paste will be affected by the moisture content of the aggregates. If these are air dry, they will absorb some water, thereby effectively lowering the *w/c* ratio and reducing the workability. On the other hand, if the aggregates are too wet, they will contribute some of their surface moisture to the paste, increasing both the *w/c* ratio and the workability and reducing strength. Therefore, these effects must be estimated and the batch weights adjusted to take them into account. The adjustments will leave the actual mix proportions unchanged on an SSD basis.

10. **Trial batch.** Having now estimated the proportions of all the ingredients, the next step is to prepare a trial batch using these estimates, using only as much water as is needed to reach the desired slump (even if it causes the permissible *w/c* ratio to be exceeded). This batch may be prepared in the laboratory or using a full-sized field batch. The concrete thus produced should be tested for slump, unit weight, yield, and air content, as well as observed for segregation tendencies and finishing characteristics. The batch weights and the unit weight are then used to calculate the weight of each material per cubic meter or cubic yard, as actually obtained in the trial batch. This is done by multiplying the weight of each constituent in the trial batch by the number of trial batches per cubic meter or cubic yard (the volume of concrete in a trial batch is the total weight of the trial batch divided by the unit weight).[8] Finally, 28-day (or other age) compressive (or flexural) strength should also be determined. Adjustments can now be made in the batch proportions for those requirements that were not satisfied by the original estimate, as follows:

a. For any deviation in slump, a new water content can be estimated based on the observation that an increase or decrease of 6 kg/m³ (10 lb/yd³) in water will increase or decrease the slump by approximately 25 mm (1 in.). If the correct slump is obtained at a lower water content, it is permissible to reduce the cement content

[8]The weight of each material per m³ or yd³ of concrete can also be calculated by multiplying the ratio of the weight of each material in the trial batch to the total weight of the trial batch by the concrete density in kg/m³ or lb/yd³ (=27 ft³/yd³ × unit weight in lb/ft³).

to reach the design *w/c* ratio, consistent with any specified limitations on cement content. If the water content must be increased to obtain the desired slump, then the *w/c* ratio will also be increased. In this case, additional cement must be added until the design *w/c* ratio is again achieved (or the entire mix redesigned).

b. If the desired amount of entrained air is not achieved, the amount of air-entraining admixture should be reestimated. The mixing water required should then be increased or decreased by 3 kg/m^3 (5 lb/yd^3) for each decrease or increase of 1% air entrainment, because of the influence of entrained air on workability.
c. If the weight method of proportioning is used and if the estimated weight of fresh concrete is incorrect, this can be reestimated from the unit weight of the trial batch, making allowance for the necessary changes in air content.
d. Any adjustment will change the yield, and therefore new batch weights must then be calculated, following the foregoing procedure from Step 5 on.

Example Mix Design

To illustrate the mix design procedure, consider the following sample problem: Concrete is required for an exterior column to be located above ground level in an area where it will be wet and subjected to substantial freezing and thawing. The concrete is required to have an average 28-day compressive strength of 30 MPa (about 4000 lb/in.^{2}).[9] For the conditions of placement, the slump should be between 75 and 100 mm (3 and 4 in.). The column is 635 mm (25 in.) square with a minimum clear space for aggregate of 50 mm (2 in.). The properties of the materials are as follows:

Cement: Type I, specific gravity = 3.15

Fine aggregate: Bulk specific gravity (SSD) = 2.63; absorption capacity = 1.3%; surface moisture = 4.2% based on SSD state; fineness modulus = 2.70

Coarse aggregate: Maximum size = 19 mm ($^3/_4$ in.); bulk specific gravity (SSD) = 2.68; absorption capacity = 1.0%; surface moisture = 0.5% based on SSD state; dry-rodded unit weight = 1600 kg/m^3 (100 lb/ft^3)

The sieve analyses of the coarse and fine aggregates fall within the limits specified in ASTM C 33. With this information, the mix design will now be carried through in detail, using the sequence of steps outlined.

Step 1: *Required material information.* This is already given.

Step 2: *Choice of slump.* The slump is also given, consistent with Table 10.1.

Step 3: *Maximum aggregate size.* The maximum aggregate size, 19 mm ($^3/_4$ in.), meets the limitations of one-fifth of the minimum dimension between forms and three-fourths of the minimum clear space.

[9]This value takes into account the difference between the average strength required and the specified design strength, which is based on the variability of the concrete (see Chapter 15).

Step 4: *Estimation of mixing water and air content.* Since the concrete will be exposed to freezing and thawing, it must be air entrained. From Table 10.2, the air content recommended for severe exposure is 6.0%; the water requirement is 180 kg/m^3 (305 lb/yd^3).

Step 5: *Water/cement ratio.* From Table 10.3, the (conservative) estimate of the required w/c ratio to give a 28-day compressive strength of 30 MPa (4500 psi) is 0.45. This does not exceed the limits based on durability in Table 10.4 (or Table 10.5).

Step 6: *Calculation of cement content.* The required cement content, based on the results of Steps 4 and 5, is 180/0.45 = 400 kg/m^3 (305/0.45 = 678 lb/yd^3).[10]

Step 7: *Estimation of coarse aggregate content.* Interpolating in Table 10.8 for the fineness modulus of the fine aggregate of 2.70, the volume of dry-rodded coarse aggregate per unit volume of concrete is 0.63. Therefore, the coarse aggregate will occupy 0.63 m^3/m^3 (or $0.63 \times 27 = 17.01$ ft^3/yd^3). The OD weight of the coarse aggregate is $0.63 \times 1600 = 1008$ kg ($17.01 \times 100 = 1701$ lb). The SSD weight is $1008 \times 1.01 = 1018$ kg ($1701 \times 1.01 = 1718$ lb).

Step 8: *Estimation of fine aggregate content.* The fine aggregate content can be established either by the mass (weight) method or by the absolute volume method.

a. *Mass (weight) method.* From Table 10.9, the estimated concrete weight is 2280 kg/m^3 (3840 lb/yd^3). Although for a first trial it is not generally necessary to use the more exact calculation based on Eq. (10.6), this value will be used here:

$$U_m = (10)(2.66)(100 - 6) + 400(1 - 2.66/3.15) - 180(2.66 - 1)$$
$$= 2264 \text{ kg/m}^3$$

Using Eq. (10.7), the equivalent value of 3812 lb/yd^3 is obtained. The weights already determined are water = 180 kg (305 lb), cement = 400 kg (678 lb), and coarse aggregate (SSD) = 1018 kg (1718 lb). Therefore, the weight of the fine aggregate (SSD) is 2264 − 180 − 400 − 1018 = 666 kg (3812 − 305 − 678 − 1718 = 1111 lb).

[10]If the problem is altered to require a 15% (weight basis) replacement of cement by fly ash, the total weight of cementitious material would be unchanged. The weight of fly ash would be $0.15 \times 400 = 60$ kg/m^3 ($0.15 \times 678 = 102$ lb/yd^3) and the weight of cement would be $0.85 \times 400 = 340$ kg/m^3 ($0.85 \times 678 = 576$ lb/yd^3). If a 15% (volume basis) replacement is used, the weight of fly ash would be $0.15\,(G_p/G_c)C_T$. For fly ash with $G_p = 2.30$, 0.15 (2.30/3.15) 400 = 44 kg/m^3 [0.15 (2.30/3.15) 678 = 74 lb/yd^3]. As with the 15% (weight basis) replacement, the cement content would be 340 kg/m^3 (576 lb/yd^3). (Volume replacement with fly ash maintains the total paste volume in the concrete, but reduces the mass percentage replacement, as well as the total weight of cementitious material.)

b. *Volume method.* Knowing the weights and specific gravities of water, cement, and coarse aggregate and knowing the air volume, we can calculate the volumes per m^3 (yd^3) occupied by the different ingredients:

Water: 180/1000 (305/62.4)	=	$0.180\ m^3$	($4.89\ ft^3$)
Cement: 400/(1000 × 3.15) [678/62.4 × 3.15]	=	$0.127\ m^3$	($3.45\ ft^3$)
Coarse Aggregate (SSD)[11]: 1018/(1000 × 2.68)	=	$0.380\ m^3$	
[1718/(62.4 × 2.68)]	=		($10.27\ ft^3$)
Air: 0.06 (0.06 × 27)	=	$0.060\ m^3$	($1.62\ ft^3$)
Total	=	$0.747\ m^3$	($20.23\ ft^3$)

Therefore, the fine aggregate must occupy a volume of $1 - 0.747 = 0.253\ m^3$ ($27 - 20.23 = 6.77\ ft^3$). The required SSD weight of the fine aggregate $= 0.253 \times 2.63 \times 1000 = 665$ kg ($6.77 \times 2.63 \times 62.4 = 1111$ lb). As may be seen, this is essentially the same as the weight calculated according to the weight method.

Step 9: *Adjustment for moisture in the aggregate.* Since the aggregates will be neither SSD nor OD in the field, it is necessary to adjust the aggregate weights for the amount of water contained in the aggregate. (Note that very dry aggregates will absorb water from the mix, and this too must be allowed for.) Only surface water need be considered; absorbed water does not become part of the mix water. For the given moisture contents, the adjusted aggregate weights become (see Section 7.1):

Coarse aggregate (wet)	= 1018 (1.005)	= 1023 kg/m^3
	[1718 (1.005)	= 1727 lb/yd^3]
Fine aggregate (wet)	= 665 (1.042)	= 693 kg/m^3
	[1111 (1.042)	= 1158 lb/yd^3]

Surface moisture contributed by the coarse aggregate is 0.5%; by the fine aggregate 4.2%. The mixing water (that needs to be batched separately) is then

$$180 - 1018(0.005) - 665(0.042) = 147\ kg/m^3$$

$$[305 - 1718(0.005) - 1111(0.042) = 250\ lb/yd^3]$$

Thus, the estimated batch weights per m^3(yd^3) are:

Water (to be added)	= 147 kg	(250 lb)
Cement	= 400 kg	(678 lb)
Coarse aggregate (wet)	= 1023 kg	(1727 lb)
Fine aggregate (wet)	= 693 kg	(1158 lb)
Total	= 2263 kg	(3813 lb)

[11]If bulk specific gravities of the aggregates are given on an OD basis, then the OD weight should be used to calculate the solid volume of the coarse aggregate, and the weight of the fine aggregate will be determined on an OD basis also.

Step 10: *Trial batch.* A trial batch is now made using the proportions calculated. The properties of the concrete in the trial batch (including unit weight) must be compared with the desired properties, and the mix design must be corrected as described. To illustrate this process, consider the following trial batch results:

Small trial batches are prepared based on the example m^3 and yd^3 mix designs ($0.015\ m^3$ and $0.02\ yd^3$, respectively). The desired slump is 100 mm (4 in.), and the desired air content is 6%. During the course of the trial match, we find that extra water is needed to achieve the desired slump. The final properties of the concrete are slump = 75 mm (3 in.), air content = 5%, and unit weight = 2286 kg/m^3 (142.6 lb/ft^3).

The weights used in the trial batches are expressed in terms of SSD aggregate:

Cement:	6.00 kg	(13.56 lb)
Coarse aggregate (SSD):	15.27 kg	(34.36 lb)
Fine aggregate (SSD):	9.98 kg	(22.22 lb)
Water (includes surface moisture on wet aggregate):	2.84 kg	(6.40 lb)
Total batch weight:	34.09 kg	(76.54 lb)

Based on manufacturers recommendations, air-entraining agent was added at a rate of 0.23 L/m^3 (6 fluid ounces/yd^3).

The batch weights and unit weight can now be used to determine the actual quantities used on a m^3 (yd^3) basis:

$$\text{Yield} = \frac{2286\ kg/m^3}{34.09\ kg/batch} = 67.06\ batches/m^3$$

$$= \left[\frac{142.6\ lb/ft^3 \times 27\ ft^3/yd^3}{76.54\ lb/batch} = 50.30\ batches/yd^3\right]$$

	m^3 batch	yd^3 batch
Cement:	6.00 × 67.06 = 402 kg	(13.56 × 50.30 = 682 lb)
Coarse Aggregate (SSD):	15.27 × 67.06 = 1024 kg	(34.36 × 50.30 = 1728 lb)
Fine Aggregate (SSD)	9.98 × 67.06 = 669 kg	(22.22 × 50.30 = 1118 lb)
Water:	2.84 × 67.06 = 190 kg	(6.40 × 50.30 = 322 lb)

Note that, due to the extra water required and lower air content, the actual weights of the ingredients differ from the original values.

The mix design must now be modified to obtain the desired slump, air content, and *w/c* ratio.

The water content for a 100-mm (4-in.) slump will be 190 kg/m^3 + 6 kg/m^3 [to increase the slump from 88 mm (3 in.)] − 3 kg/m^3 [to take into account the extra slump that will be obtained as the 5% actual air content is increased to the desired 6%] = 193 kg/m^3 (322 + 10 − 5 = 327 lb/yd^3).

We now proceed from Step 5 to recalculate the batch weights.

Step 5: w/c = 0.45 is unchanged

Step 6: Cement content = 193/0.45 = 429 kg (327/0.45) = 727 lb

Step 7: Coarse aggregate (SSD) content = 1018 kg (1718 lb) is unchanged

Step 8: Fine aggregate (SSD) content: For this problem, we will use the "volume" method.

Water: 193/1000 (327/62.4)	=	0.193 m^3	(5.24 ft^3)
Cement: 429/(1000 × 3.15) [727/(62.4 × 3.15)]	=	0.136 m^3	(3.70 ft^3)
Coarse Aggregate (SSD): 1018/(1000 × 2.68)	=	0.380 m^3	
[1718/(62.4 × 2.68)]	=		(10.27 ft^3)
Air: 0.06 (0.06 × 27)	=	0.060 m^3	(1.62 ft^3)
Total:	=	0.769 m^3	(20.83 ft^3)

Absolute volume of fine aggregate = 1 − 0.769 = 0.231 m^3 (27 − 20.83 = 6.17 ft^3). SSD weight of fine aggregate = 0.231 × 2.63 × 1000 = 608 kg (6.17 × 2.63 × 62.4 = 1012 lb).

Air-entraining agent. As discussed in Chapter 8, the amount of air entrained is a function of other properties of the concrete, such as temperature, slump, and aggregate gradation. The two factors with the greatest effects are the amount of air-entraining agent (as one might expect) and the amount of cement (increased cement content tends to decrease the amount of air entrained). The new dosage rate of air-entraining agent is calculated by multiplying the original dosage by the ratios of (1) desired air content to actual air content and (2) new cement content to initial cement content:

$$\text{Air-entraining agent} = 0.23\,(6/4)\,(429/400) = 0.37\ \text{L/m}^3$$

$$= [6\,(6/4)\,(727/678) = 9.7\ \text{fl. oz/yd}^3]$$

A new trial batch should now be made to check the concrete properties. In addition to slump, air content, and unit weight, the concrete should be checked for cohesiveness, finishability, and strength.

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PROBLEMS

10.1 What are the criteria that must be used when designing a concrete mix?

10.2 Calculate Abrams' w/c ratio law for the w/c ratios given in Table 10.3 for non-air-entrained concrete and compare the calculated strengths with those tabulated.

10.3 Plot the ideal grading curves computed from Eq. (10.3) with q = 0.33, 0.5, and 0.67 for a maximum aggregate size of 25 mm. Compare your results with the ASTM grading limits.

10.4 Design an air-entrained concrete mix using a maximum aggregate size of 27 mm to be used for an interior beam having a 28-day compressive strength of 35 MPa. The material properties are as follows:
Cement: All types have a specific gravity of 3.15.
Coarse aggregate: Bulk specific gravity (SSD) = 2.90, absorption capacity = 0.5%, effective absorption = 0.3%, and dry-rodded unit weight = 1700 kg/m^3.
Fine aggregate: Bulk specific gravity (SSD) = 2.62, absorption capacity = 1.2%, surface moisture = 2.8%, and fineness modulus = 2.80.
Use the upper value of the recommended slump range. Present the results based on the given moisture contents.

10.5 Recompute the mix in Problem 10.4 if the bulk specific gravities of the aggregate now represent the OD condition and if the total moisture contents are 0.8% (coarse) and 2.5% (fine).

10.6 Recompute the mix in Problem 10.4 using a 15% *weight* replacement of cement by fly ash with a specific gravity of 2.2. Present the results in terms of SSD aggregate.

10.7 Design a concrete mix that has a 28-day compressive strength of 4500 lb/in.^{2} and a maximum aggregate size of $^3/_4$ in. The concrete will be placed in a column exposed to freezing and thawing and will be in contact with soil that has a sulfate content of 0.3%. The material properties are as follows:
Cement: All types have a specific gravity of 3.15.
Coarse aggregate: Bulk specific gravity (SSD) = 2.65, absorption capacity = 1.5%, surface moisture = 1.0%, and dry-rodded unit weight = 105 lb/ft^3.
Fine aggregate: Bulk specific gravity (SSD) = 2.75, absorption capacity = 1.0%, surface moisture = 3.0%, and fineness modulus = 2.70.
Use the upper value of the recommended slump range. Present the results based on the given aggregate moisture contents.

10.8 Recompute the mix in Problem 10.7 if the bulk specific gravities of the aggregates now represent the OD condition and if the total moisture contents are 1.0% (coarse) and 3.0% (fine).

10.9 Recompute the mix in Problem 10.7 using a 10% *volume* replacement of cement by fly ash with a specific gravity of 2.4. Present the results in terms of SSD aggregate.

10.10 The following batch weights (cubic meter, SSD basis), selected using ACI 211.1, produced concrete with a unit weight of 2280 kg/m^3, a slump of 75 mm, and an air content of 5%.
Cement: 408 kg/m^3 (specific gravity 3.15).
Water: 163 kg/m^3.
Coarse aggregate: 1018 kg/m^3 [19 mm, bulk specific gravity (SSD) = 2.62, absorption capacity = 3.8%, dry-rodded unit weight = 1505 kg/m^3].
Fine aggregate: 653 kg/m^3 [bulk specific gravity (SSD) = 2.58, absorption capacity = 1.0%, fineness modulus = 2.50].
Air-entraining agent: 0.30 L/m^3.
a. What volume of concrete was produced?
b. What were the actual mix proportions on a cubic meter basis?
c. What batch quantities (cubic meter, SSD basis) should be used to produce concrete with a 100-mm slump and $6\frac{1}{2}$% air content?

10.11 The following batch weights (cubic yard, SSD basis), selected using ACI 211.1, produced concrete with a unit weight of 141.8 lb/ft^3, a slump of 2 in., and an air content of 4%.
Cement : 573 lb/yd^3 (specific gravity 3.15).
Water : 275 lb/yd^3.
Coarse aggregate: 1657 lb/yd^3 [$\frac{3}{4}$ in., bulk specific gravity (SSD) = 2.58, absorption capacity = 3.5%, dry-rodded unit weight = 94 lb/ft^3].
Fine aggregate: 1247 lb/yd^3 [bulk specific gravity (SSD) = 2.62, absorption capacity = 1.5%, fineness modulus = 2.70].
Air-entraining agent: 8 fl. oz/yd^3.
a. What volume of concrete was produced?
b. What were the actual mix proportions on a cubic yard basis?
c. What batch quantities (cubic yard, SSD basis) should be used to produce concrete with a 4-in. slump and $6\frac{1}{2}$% air content?

CHAPTER 11

Concrete Construction Practices

The production of high-quality concrete does not rest solely on proper proportioning. The concrete placed in a structure must be of uniform quality, free of voids and discontinuities, and adequately cured. Although the design of a concrete mix is critical for strength and durability, lack of sufficient attention to mixing, handling, and placing can result in poor concrete from a well-designed mix. In this chapter, we discuss batching and mixing and the various methods of transporting, placing, consolidating, and finishing concrete. Curing methods are the subject of Chapter 12. The overall objective is to ensure that the concrete within a structure is a uniform blend of the constituent materials in the correct proportions and thus conforms to the specifications.

11.1 BATCHING AND MIXING

Batching

Batching of aggregates and cements is best done by weight, since dispensing of solids on a volume basis can lead to gross errors. Only water and liquid admixtures can be measured accurately by volume. Batching by weight also allows rapid and convenient adjustment of batch weights of aggregate and water when changes in aggregate moisture contents occur. Environmental and other practical considerations often make it desirable to retain some wash water in the mixer. Therefore, this water must also be accounted for during the batching operation. Material quantities should be measured with a high degree of accuracy. The tolerances that are frequently used are given in Table 11.1, which is based on ASTM C 94, Specifications for Ready-Mixed Concrete. Weighing equipment should be capable of measuring quantities within these tolerances for the smallest batches that will be used. On small jobs, cement can be dispensed by sack (fractional sacks should be weighed), although the tolerance allowed by ASTM on individual sacks in the United States (3%) is greater than that recommended in Table 11.1.

TABLE 11.1 Recommended Tolerance for Batching Concrete Constituents[a]

	Individual[b]	*Cumulative*[b]
Cement	±1%	±1%
Water	±1%	±3%
Aggregates	±2%	±1%
Admixtures	±3%	N.R.[c]

[a] Batch weights should be greater than 30% of scale capacity.
[b] Individual refers to separate weighing of each constituent. Cumulative refers to cumulative weighing of cement and pozzolan of fine and coarse aggregate, or water from all sources (including wash water).
[c] Not recommended except for pozzolans.

Handling Cementitious Materials Cement and pozzolans must be kept dry to prevent the initiation of hydration and difficulties in handling. Each type of cementitious material must be stored and batched separately, and handling equipment must be designed to prevent packing or clogging.

Bags of cement should be stored on pallets and protected from moisture. To prevent handling difficulties caused by "warehouse pack" or "pack set," bags should not be stacked more than 14 high for periods of up to 60 days or 7 high for longer periods. If warehouse pack does occur, the bags can still be used, if the cement has remained dry, by rolling them over several times to break up the packed cement. Warehouse pack can be distinguished from hydration by squeezing the packed cement between the fingers. Cement that has undergone warehouse pack can be broken into a powder, with no gritty particles, while cement that has become wet cannot.

Handling of Aggregates In Chapter 7, the importance of proper sampling was emphasized to obtain aggregate properties from a truly representative sample. However, proper sampling will be of little value if suitable precautions are not also taken to prevent segregation of aggregate sizes during handling. Figure 11.1 indicates some of the proper ways to handle aggregate to avoid segregation. In some plants, the division of coarse aggregate into coarse, medium, and fine fractions can minimize this problem. The proper gradation is then achieved during batching. Segregation of fine aggregate can be limited by keeping it wet. Variations in the gradation of fine aggregate that cause a change in the fineness modulus greater than ±0.2% require a modification in the mix proportions. Aggregate gradation should be checked at least on a daily basis. Protection of aggregates from the weather and providing adequate drainage will eliminate gross variations in aggregate moisture contents. For good batching control, proper allowance for changes in the moisture content should be made. The variations are greatest for the fine aggregate (see Chapter 7), and often equipment for monitoring the moisture content of the sand (and occasionally the smallest coarse aggregate-size fraction) is incorporated into an automatic batching plant. Such equipment should be regularly checked by standard moisture content tests.

Batching Equipment Any batching equipment that operates reliably within the specified tolerances can be used, provided that it minimizes segregation by allowing free,

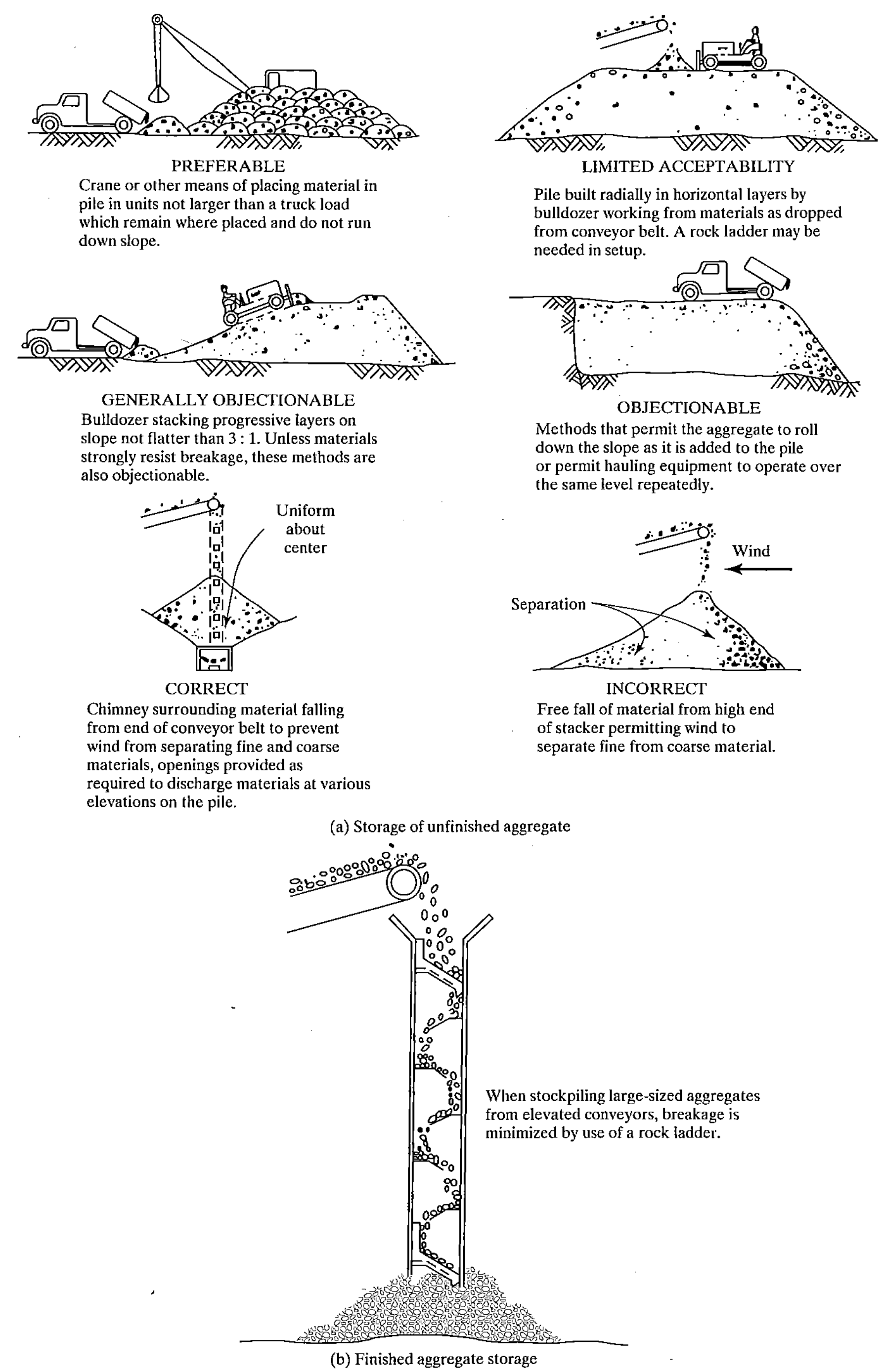


FIGURE 11.1

Handling and storing aggregates: (a) storage of unfinished aggregate; (b) finished aggregate storage. [Adapted from ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-00)," *ACI Manual of Concrete Practice*, Part 2. American Concrete Institute, Farmington Hills, MI (2001).]

unobstructed flow of materials. The exact arrangement and type of equipment can vary widely, but for each type of material there should be a storage bin that discharges into a weight batcher and then to the mixer. Typical recommended arrangements are shown in Figure 11.2. Cement is preferably batched separately, but aggregates may be batched separately or cumulatively.

Available materials-handling equipment can be divided into three general categories: manual, semi-automatic, and fully automatic. Manual batching is generally only acceptable for small jobs (up to 400 m^3 or 520 yd^3) and low output requirements (15 m^3/h or 20 yd^3/h); otherwise, semiautomatic or automatic batching should be used. In semiautomatic arrangements, the charging and discharging of the batchers are activated manually, but are automatically terminated. In a fully automatic system, a single starter switch activates the batching sequence. Both systems require interlocks to maintain tolerances and to prevent the batcher from discharging and charging simultaneously. Automatic systems, when properly designed, can maintain high-speed batching within specified tolerances.

Batching equipment should be kept clean and receive regular maintenance. The accuracy of the weight batchers or other measuring devices should be checked regularly. This is particularly important when batching finely divided solids (cement and pozzolans), since dust from these materials can cause malfunction of the weighing mechanism if it is not properly protected. Avoiding free fall of cement avoids excessive dust and material loss. Proper transfer of such fine material can be a problem, and vibration or aeration are often used to facilitate complete discharge. Dispensing equipment for admixtures requires more frequent maintenance and calibration. Malfunctions can easily lead to severe overdoses, which can cause serious problems with the fresh or hardened concrete. Liquid dispensers should be regularly flushed with water to prevent buildup of gummy deposits or sediments that could clog or stick valves and damage pumps.

Mixing

Thorough mixing is essential for the complete blending of the materials that are required for the production of homogeneous, uniform concrete. Not only does inadequate mixing result in lower strengths, but also in greater batch-to-batch and within-batch variations. However, overly long mixing times do not improve the quality of concrete and may severely limit the output of the batching plant. Unusually long mixing times may cause some breakdown of the aggregate and decrease the air content.

Mixing Times The optimum mixing time depends on (1) the type of mixer, (2) the condition of the mixer, (3) the speed of rotation, (4) the size of the charge, and (5) the nature of the constituent materials. Thus, the most efficient mixing time should be assessed in the field by determining batch-to-batch variations using the materials and mixing conditions for the job. Lean, dry, or harsh mixes require longer mixing times; concretes made with angular aggregates need more mixing than those made with rounded gravels. A good rule of thumb for minimum mixing time (once the mixer is fully charged) is 1 min of mixing time for 1 m^3 of concrete plus $\frac{1}{4}$ min for each additional 1 m^3.

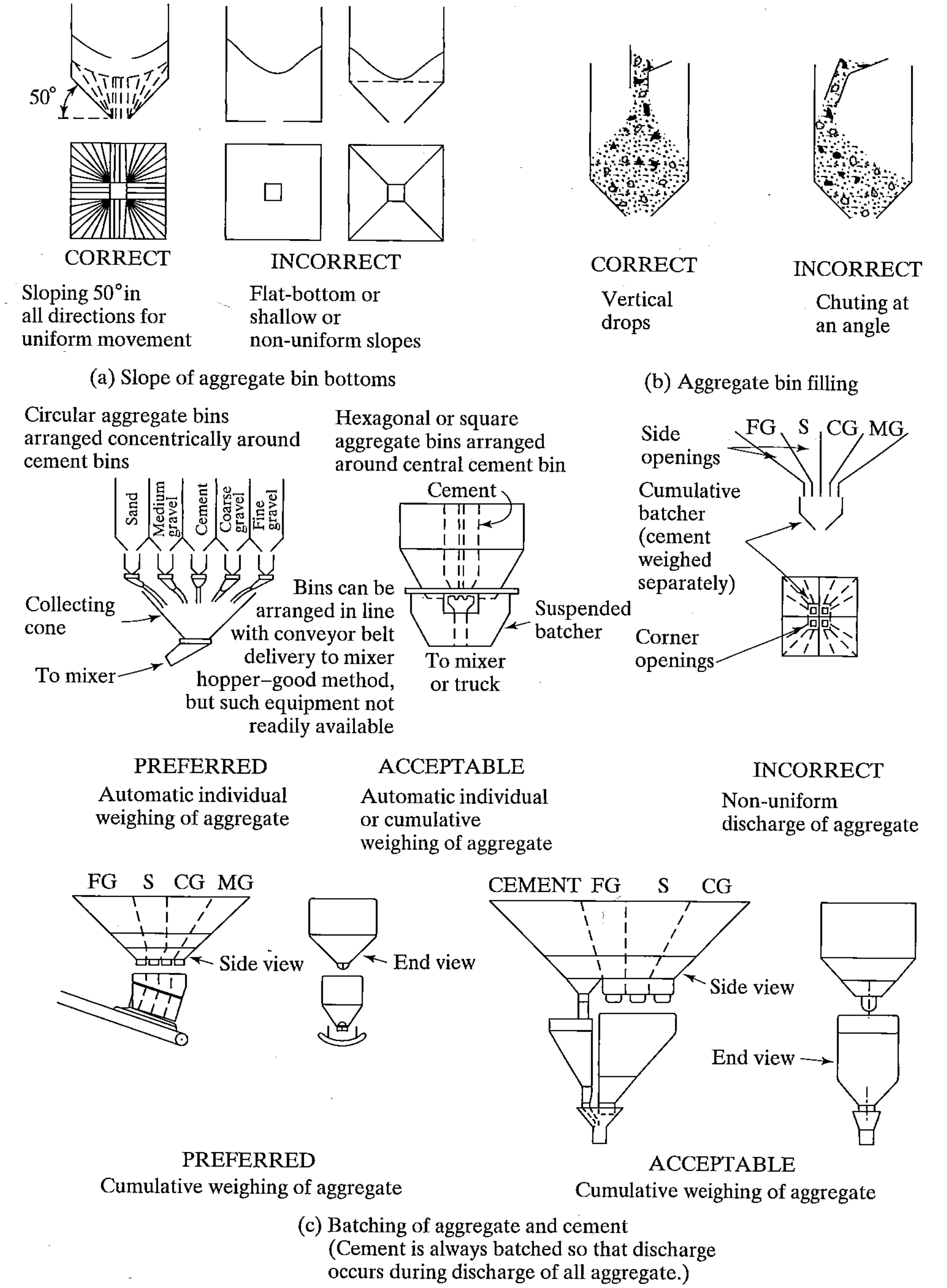


FIGURE 11.2

Methods of batching: (a) slope of aggregate bin bottoms; (b) aggregate bin filling; (c) batching of aggregate and cement. [Adapted from ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-00)," *ACI Manual of Concrete Practice*, Part 2. American Concrete Institute, Farmington Hills, MI (2001).]

In practice, excess mixing times (usually due to unexpected delays or poor planning) are encountered more often than inadequate mixing. Most construction specifications place a maximum limit on the period between the initiation of mixing and the discharge of the concrete. Rarely does the limit take into account the effect of concrete temperature on the rate at which the concrete will set. High concrete temperatures greatly limit the working time, while low concrete temperatures provide extended periods during which the concrete can be placed, consolidated, and finished. A good rule of thumb for concrete temperatures between 32°C (90°F) and 4°C (40°F) is a maximum mixing time of 1 h at a concrete temperature of 24°C (75°F) plus (or minus) 15 min for each 2.8°C (5°F) increase (decrease) in concrete temperature. Thus, concrete can be mixed for up to $2\frac{3}{4}$ h at 4°C (40°F), but for only 15 min at 32°C (90°F). At 35°C (95°F), a practical upper limit, concrete can be mixed for just 10 min. Establishing maximum mixing times based on concrete temperature can serve as an excellent tool for maintaining concrete quality.

Charging the Mixer The complete mixing cycle includes the charging and discharging process, so that its duration may easily be two to three times that of actual mixing. Charging of the mixer should be an opportunity to preblend the materials. Although the sequence can vary to suit particular applications, it is desirable to add about 10% of the mixing water before the aggregates are added. The water should enter well inside the mixer and be added uniformly during the whole time the solid ingredients are added, leaving about a final 10% to be added at the end. The cement should enter the mix after about 10% of the aggregates have been charged. Preblending is important for truck mixers with tilted drums, which can produce highly variable concrete within a single batch, especially when the cement is added last. Simultaneous charging of the solid ingredients after the first 10% of the aggregate is added will minimize uniformity problems. Mineral admixtures are generally added with the cement, but water-soluble admixtures should be dissolved in the mixing water. If more than one water-soluble admixture is used, they should be batched separately, not premixed, and added to each batch at the same time in the mixing sequence and in the same order. By allowing one admixture to interact with the solid ingredients before the second is added, adverse interactions between the two admixtures may be avoided.

Types of Mixers A great variety of concrete mixers are available on the market today. These can generally be divided into three categories: drum mixers, pan mixers, and continuous mixers. The capacity of such mixers can range up to 9 m³ (12 yd³). Laboratory mixers are designed to handle as little as 0.003 m³ ($\frac{1}{10}$ ft³), while planetary mixers are recommended for mixing mortars (ASTM C 305.)

A satisfactory type of *drum mixer* has an arrangement of interior fixed blades to ensure end-to-end exchange of material during mixing. In some cases, a horizontal shaft with spiral blades may be rotated in the drum. Tilting drums, which are commonly used for small jobs, are also available in large sizes. They have the advantage of a quick, clean discharge even of dry mixes. Horizontal drums are discharged by inserting a chute to intercept the concrete as it is mixing or by reversing the direction of rotation, which forces the concrete out of the mixer. Some *slipform pavers* have drum-type mixers attached to a unit that spreads the concrete by means of a screw and also screeds and vibrates the concrete after it has been placed.

Pan mixers are particularly good for mixing lean and dry mixes. They are thus commonly used in precast concrete plants, where their greater bulk and less convenient discharge are not necessarily disadvantages. The blades are fixed to an assembly so that they agitate the concrete throughout the stationary pan as the vertical drive shaft rotates. In another type, an eccentric mounting of blades revolves within a rotating pan. Small-scale pan mixers are popular for trial mixing in the laboratory, as they produce consistent concrete throughout a wide range of mixes.

In a *continuous mixer*, the materials are fed into a mixing trough by means of conveyors. The concrete is mixed by a spiral blade as it passes through the trough to the discharge end. The concrete is proportioned by adjusting the relative speeds of the conveyors. Thus, proportioning is actually by volume, so that it is more difficult to produce uniform concrete, unless the aggregate source is uniform and the moisture content of the sand is kept constant to avoid errors introduced by bulking (Section 7.1). The conveyors should be well maintained to ensure an accurate supply of materials. Continuous mixers are useful where a continuous supply of concrete is desired, such as mass concrete construction with roller compacted concrete (discussed later in this chapter). Portable continuous mixers provide a convenient means to produce small quantities of concrete over an extended period.

Ready-Mixed Concrete The vast majority of concrete in North America is placed using concrete mixed at permanent batching plants, which are found in virtually all communities with a sizable population. Ready-mixed concrete has advantages for both small and large jobs. The advantages are to be found not only in automated equipment and permanent, trained personnel, which result in better quality control, but also in the elimination of materials storage on congested building sites. Since the batch plant must be within reasonable distance of the building site, field batch plants must still be used in more remote locations. There are several ways in which concrete from central batching plants can be handled. These are central-mixed, transit-mixed, shrink-mixed, and truck-mixed.

Central-mixed concrete is completely mixed at the plant, and the truck mixer is only used as an agitating conveyance. Slight agitation of the concrete during transportation reduces the amount of slump loss that occurs when the concrete is left standing, and prevents segregation. The truck can also be used to remix the concrete, if necessary, at the job site. *Transit-mixed* concrete is partially or completely mixed during the time the concrete is being transported to the job site. *Shrink-mixed* concrete is partially mixed at the plant to shrink or reduce the overall volume, and mixing is completed in the truck mixer.

Truck-mixed concrete is completely mixed within the mobile mixer after it has been charged at the central plant. The advantage of truck mixing is that the water can be kept separate from the solid materials and mixed just prior to placement at the construction site. Therefore, problems of delays in transportation or placement are avoided. Of course, the aggregates must be surface dry, or some cement hydration will start once the cement comes in contact with the aggregates. In most cases, however, the concrete is

partially or totally mixed in transit, with mixing completed at the job site, as required. The amount of concrete that can be transported in a truck mixer depends on the method of mixing. To provide proper agitation, the volume of central-mixed concrete is limited to 80% of the total volume of the truck mixer drum. For all other methods of batching, the concrete quantity is limited to 63% of the drum volume, since the drum must serve as a mixer as well as an agitator. ASTM C 94 requires the concrete to be placed within $1\frac{1}{2}$ h of mixing or before the drum has passed through 300 revolutions, but allows these limits to be waived by the purchaser if the concrete is workable enough to be placed without adding water to the batch. ASTM C 94 allows the purchaser to specify other mixing times when quick stiffening of concrete is a concern.

Remixing When ready-mixed concrete is supplied at the job site, it is often remixed just before placing to ensure that the correct slump is achieved. If this is the case, the earlier period of mixing should be such that at least half the minimum mixing time occurs during the remixing period and that the total number of revolutions does not exceed specifications. It is common practice to add materials (particularly water) to the concrete after it has been batched, usually at the job site, to ensure that the specified properties are attained before placement. With the exception of superplasticizers, which are regularly added at the job site to obtain the desired level of workability, adding materials at the job site should be discouraged, since it will only lead to trouble. Accurate batching of most other materials on the job site is difficult, so that quality control suffers. The concrete mix should be designed to ensure that the correct properties are attained at the time of placement. This means that normal slump loss must be allowed for by designing for a higher slump than that specified for placement.

11.2 TRANSPORTATION

There are many different ways of handling concrete, and the choice will depend mostly on the amount of concrete involved, the size and type of construction, the topography of the job site, the location of the batch plant, and the relative costs. From a materials point of view, any method of transportation should protect the concrete from the effects of the weather (heat, cold, or moisture). It should also not cause undue segregation by excessive jarring or shaking. In short, the methods used should be selected to maintain concrete quality, which should not be lowered to accommodate available equipment. Methods of transportation can be roughly assigned to four general categories (see Table 11.2): wheeled transports, buckets, conveyors and chutes, and pumps.

Transport of concrete from the batching plant to the job is generally by ready-mixed trucks equipped with a revolving drum for agitation. The use of dumpsters should not involve long trip times to avoid excessive slump loss or segregation. Buckets, overhead cableways, or trucks can be used to transport concrete from an adjacent batch plant.

Buckets or skips are a versatile means of moving concrete about the job site by the use of a hoist, crane, or overhead cableway. Large quantities [up to 6 m^3 (8 yd^3) at a time] can be moved horizontally or vertically. Buckets should be designed so that discharge can be properly regulated and so that segregation does not occur during discharge. The bucket should slope down to the exit gate and the concrete should be

TABLE 11.2 General Methods of Transporting Concrete

Method	*Application*	*Capacity*[a]	*Remarks*
Truck (agitated)	Ready-mixed concrete	2–9 m^3	Up to $1\frac{1}{2}$ h trip time
Truck or rail min trip time; cars (nonagitated)	From nearby batch plant to job site		30–45 avoid wet mixes or very dry mixes (slump range 12–50 mm or ½–2 in.)
Buckets or skips	On site, can be transferred from nearby batch plant	Up to 6 m^3	Requires cranes or hoists
Power buggies	On site	About 0.3 m^3	Requires good terrain;
Manual buggies		About 0.2 m^3	vertical transfer limited
Belt conveyors	On site	Up to 340 m^3/h	Provides maximum capacity,[a] avoid wet mixes; use a cohesive mix, 50–100 mm (2–4 in.) slump
Pumping	On site	Up to 120 m^3/h	Mixes should be designed especially for pumping

[a] m^3 × 1.32 = yd^3.

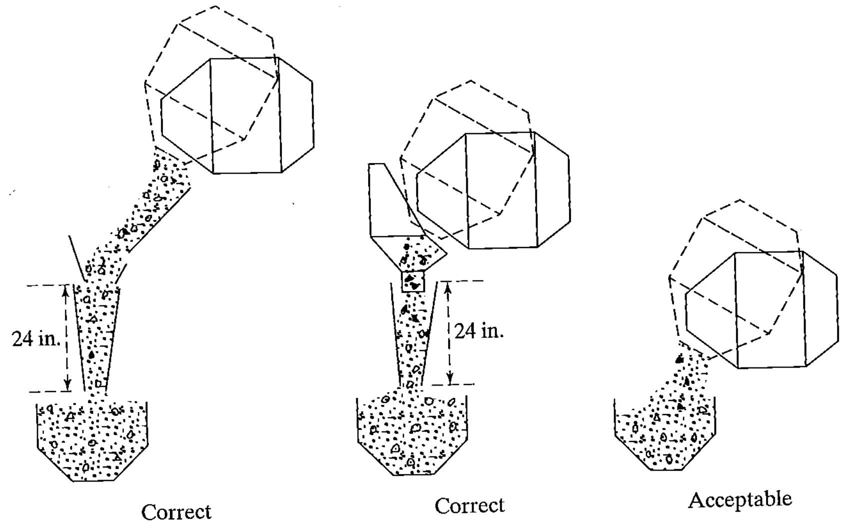
released vertically (Figure 11.3). The bucket should be charged from the mixer in a similar fashion.

Manual or motorized buggies can be used to transport small loads around the job site on good terrain. They can only be used to advantage on level ground or slight grades.

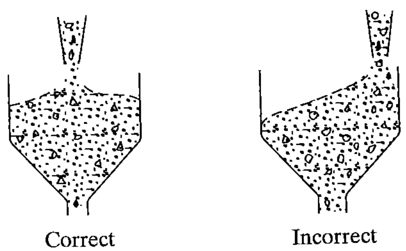
Belt conveyors can also be used to transport concrete both horizontally and vertically, but special care should be taken with this form of transportation. Segregation is liable to occur at transfer points between conveyors or during discharge. Wet or lean mixes are liable to segregate particularly if the incline of a conveyor is too great. A large maximum aggregate size should be avoided. Care should also be taken to prevent mortar from adhering to the belt; it should be scraped off at the point of discharge. Since the concrete is spread out relatively thinly, it is susceptible to loss of moisture or increase of temperature during hot weather, which will accelerate slump loss. Thus, belts should be protected from direct sunlight and wind, and their use is not recommended in very hot conditions. Chutes can be used when the concrete is being placed at points below the point of delivery. They suffer from the same disadvantages.

Pumping

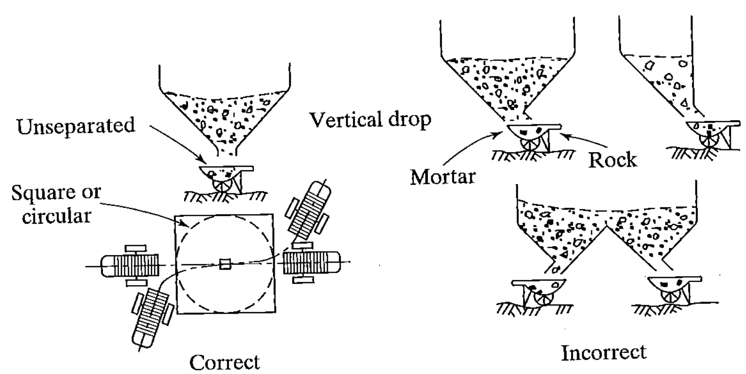
Pumped concrete is conveyed under pressure through a rigid pipe or flexible hose and deposited at the desired point within a construction site. The method is very versatile and is particularly useful where space is limited, such as in tunnels. Concrete is regularly pumped over quite large distances, typically more than 450 m (1500 ft) horizontally



(a) Discharging mixers



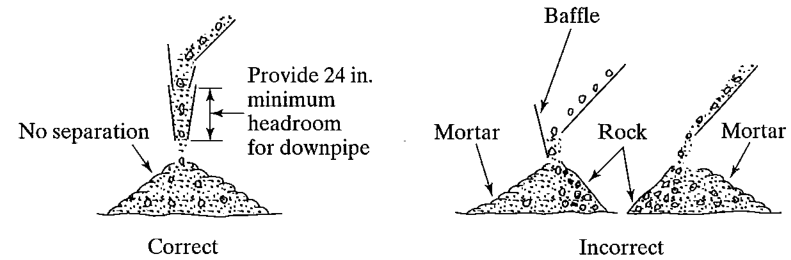
(b) Filling hoppers or buckets



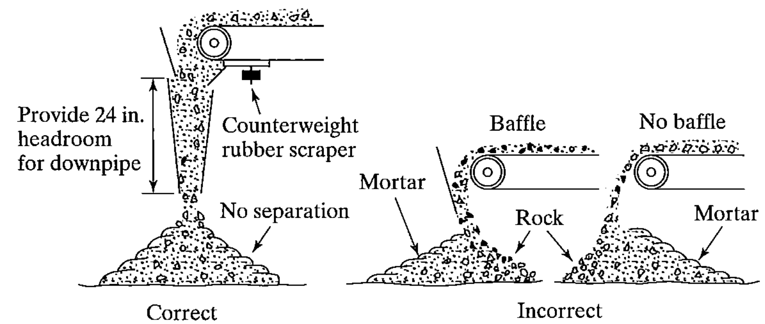
(c) Discharge of hoppers for loading buggies

FIGURE 11.3

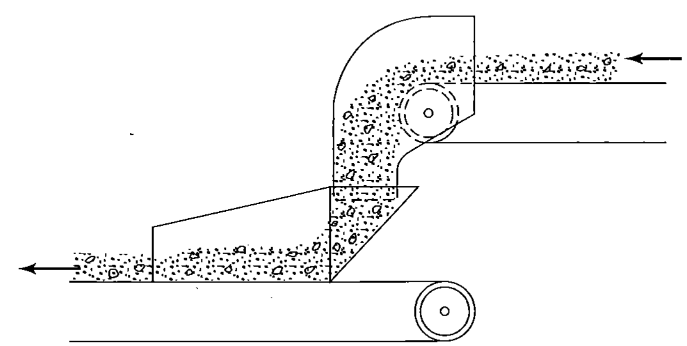
Handling of concrete: (a) discharging mixers; (b) filling hoppers or buckets; (c) discharge of hoppers for loading buggies; (d) control of separation at the end of concrete chutes; (e) control of separation of concrete at the end of a conveyor belt; (f) control at transfer point of two conveyor belts. [Adapted from ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-00)," ACI Manual of Concrete Practice, Part 2. American Concrete Institute, Farmington Hills, MI (2001).]



(d) Control of separation at the end of concrete chutes



(e) Control of separation at the end of a conveyor belt



(f) Control at transfer point of two conveyor belts

FIGURE 11.3 (d)–(f)

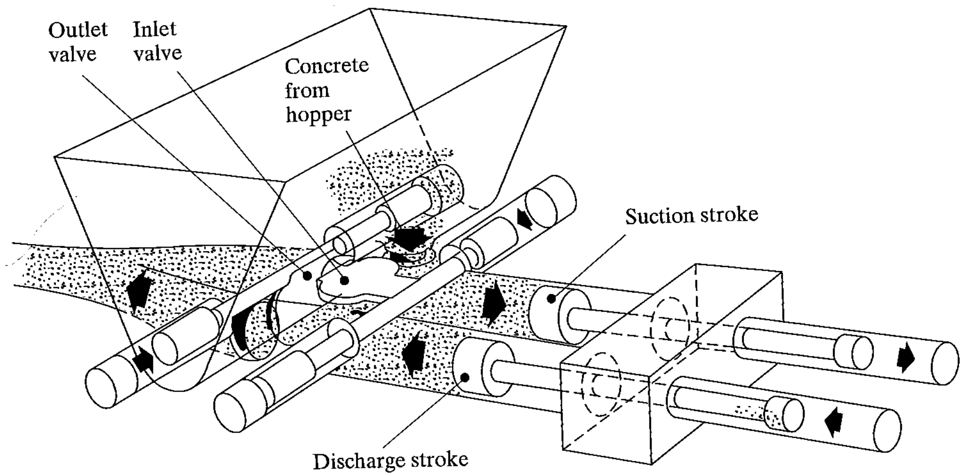


FIGURE 11.4
Concrete pump. [From T. H. Cooke, *Concrete Pumping and Spraying*, Thomas Telford, London (1990).]

and 150 m (500 ft) vertically, and special high pressure pumps have delivered concrete to distances as great as 1400 m (4600 ft) horizontally and 420 m (1400 ft) vertically.

Pumping Equipment The principal type of concrete pump used in North America is a piston pump (see Figure 11.4). The pump operates with two pistons and synchronized inlet and outlet valves. For each piston, on the backward stroke the inlet valve opens and concrete is drawn into the cylinder from the hopper, while on the forward stroke the concrete is forced into the pipeline through the open outlet valve. The pistons move in opposite directions, providing a steady flow of concrete. The pistons and valves are usually operated hydraulically, with power provided by a gasoline or diesel engine or an electric motor.

Concrete pipelines can be made either of rigid pipe or heavy-duty flexible hose. The latter does not perform as well as rigid pipe because it presents a greater resistance to the movement of concrete, but it is used in rigid lines for curves and is invaluable for use with moving booms and other situations where flexibility is required. Generally, flexible hose greater than 125 mm (5 in.) i.d. is not used due to difficulties in handling, but rigid pipe up to 200 mm (8 in.) i.d. is common. The latter must be moved by mechanical means on the job site. Small-bore pumps are used for mobile pumping units that can be moved rapidly from site to site and will conveniently handle relatively small quantities of concrete. Such units are usually equipped with a hydraulically operated boom for both vertical and horizontal placements. With a 100 to 125-mm (4 to 5-in.) line, these units can deliver 30 to 60 m^3/hr (40 to 80 yd^3/hr) up to 360 m (1200 ft) horizontally and 90 m (300 ft) vertically.

Pipe or hose materials should be resistant to wear and abrasion, should be reasonably lightweight, and should not react with concrete. Thus, aluminum, because of its reactivity with alkalis, is not recommended as a pipeline material unless satisfactory

performance can be demonstrated. Couplings between pipe sections should be watertight, strong enough to withstand stresses from misalignment or poor pipe support, present no obstruction to concrete flow, and be easily connected or disconnected.

Pumping Distances The distance concrete can be pumped depends on many job factors: (1) the capacity of the pump, (2) the size of the pipeline, (3) the number of obstructions to uniform flow, (4) the velocity of pumping, and (5) the characteristics of the concrete. The pump must supply enough force to overcome friction between the concrete and the interior surface of the line. A knowledge of actual frictional losses can be used with conventional hydraulic theory to calculate pump capacity for a given rate of delivery. Bends in the line and decreases in pipe diameter considerably add to frictional resistance. When concrete is pumped vertically, extra force is needed to overcome gravity, requiring about 23-kPa/m (1-lb/in.^{2}/ft) lift. Proper maintenance of pipe and couplings is needed to minimize frictional losses.

Concrete moves as a cylindrical plug when it is pumped, separated from the pipe wall by a thin lubricating film of mortar or grout. Thus, the line should be initially primed by starting pumping with a properly designed mortar. Lubrication can also be supplied by concrete containing 60 to 120 kg/m^3 (100 to 200 lb/yd^3) more cement than the regular concrete or using the regular concrete with the coarse aggregate removed. For 150 to 200-mm (6 to 8-in.) horizontal lines, 0.4 m^3 (0.5 yd^3) will lubricate about 300 m (1000 ft) of line, and this lubrication will be maintained as long as pumping continues. Once pumping is completed, the pipes are cleaned by blowing compressed air through and then flushing with water until completely clean.

Mix Design Special attention should be paid to mix design when concrete is to be pumped. Concrete for pumping is not radically different from other concrete, but it should be plastic and cohesive; harsh or dry mixes do not pump well. More emphasis on quality control is needed, and materials should be uniform in properties throughout the job. The two major causes of failure to pump concrete successfully are high frictional resistance and a tendency to segregate. As a result, badly proportioned concrete requires a greater pump capacity and is prone to pipe blockage.

The ratio of the maximum size of the coarse aggregate to the smallest inside pipe diameter should not exceed 0.33 for angular aggregates, or 0.40 for well-rounded gravel, to minimize the possibility of blockages. The amount of coarse aggregate is usually reduced (by up to 10%) compared to mixes that are not pumped (see footnote to Table 10.8), but for well-proportioned mixes, used with good quality control and simple pipeline layouts, this may not be necessary. Size gradation of both fine and coarse aggregate should be as close as possible to the middle range of the grading limits given in ASTM C 33, but uniformity of gradation is even more important. Blending of size fractions may be needed to achieve this. The role of the fine aggregate is particularly important, since it is the fluid mortar that is the pumping medium in which the coarse aggregate is suspended. Special attention should be given to having adequate fines; designation of fineness modulus alone is not enough. For small line systems [$<$125 mm (5 in.)], 15 to 30% should pass the 300-μm (No. 50) sieve and 5 to 10% should pass the 150-μm (No. 100) sieve. Lightweight aggregates, both fine and coarse, should be

thoroughly presoaked prior to use to avoid excessive slump loss in the pipeline. Because of high absorption, even after presoaking, the coarse aggregate content of lightweight mixes may need to be reduced by up to one-third compared to a nonpumped mix. However, if vacuum or thermal saturation is used, the lightweight mixes can be proportioned using the same methods as normal-weight concrete. Pumping aids are available commercially; these are water-soluble polymers that reduce frictional resistance and bleeding in the pipeline by increasing the viscosity of water. The effectiveness of such admixtures for a particular job should be established by on-site evaluation.

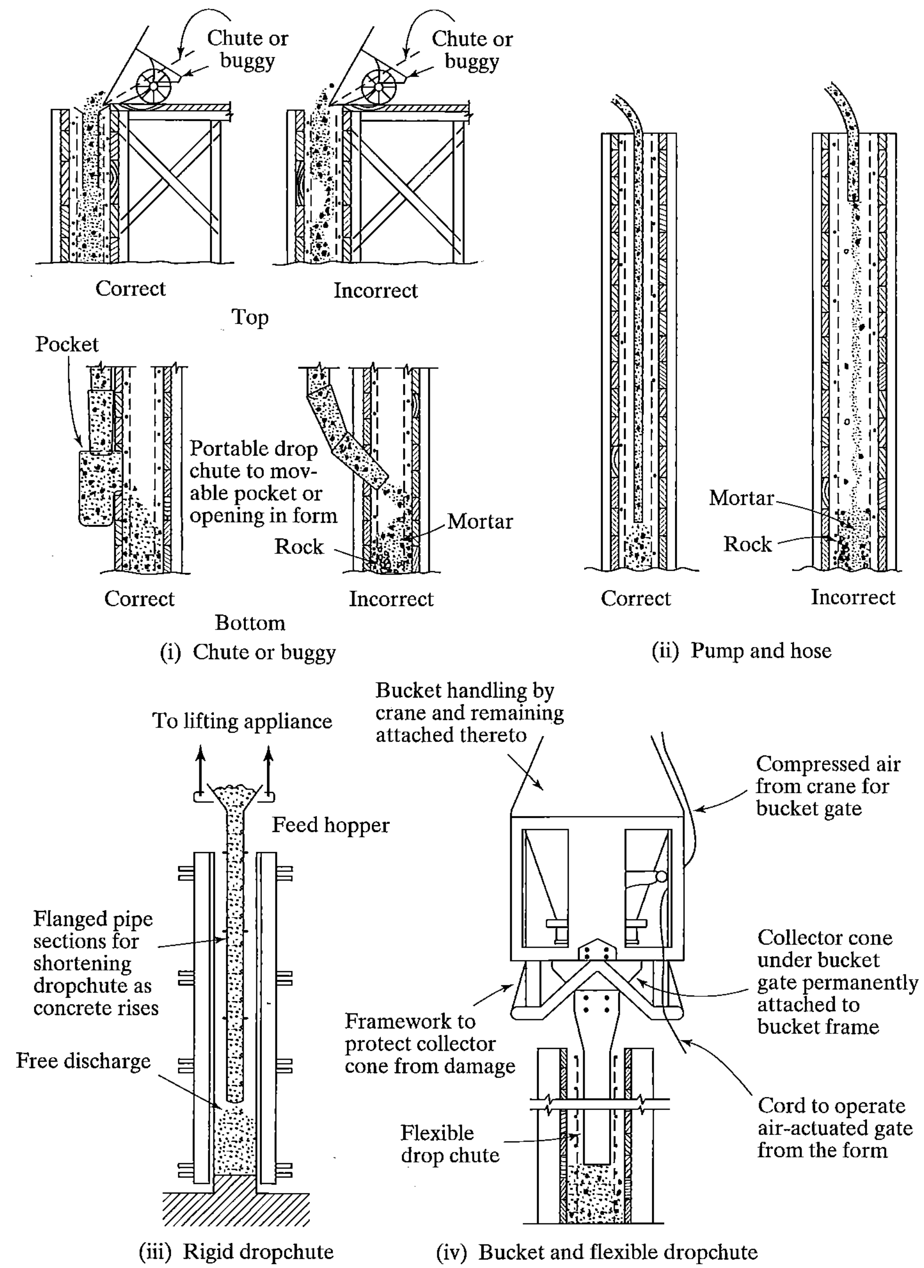
Concrete with slumps in the range 50 to 150 mm (2 to 6 in.) can be successfully pumped if properly proportioned. High slumps may cause segregation of coarse aggregate and excessive bleeding, which can lead to loss of lubrication and blocking of the lines. Adding water solely to correct slump may cause more problems by lowering the cohesiveness of the concrete. The desired slump should be achieved by proper proportioning. This will usually require additional cement because of the increased fine aggregate content, but mineral admixtures or water-reducing admixtures can be used to reduce cement requirements. Air-entrained concrete is more cohesive, more workable, and less prone to segregation and bleeding than non-air-entrained concrete and is generally easier to pump. Air contents should be measured at the point of discharge, since pumping can alter the air void system.

11.3 PLACEMENT OF CONCRETE

Proper handling of concrete during placement in forms can prevent problems of segregation of the coarse aggregate that can result in the formation of rock pockets and honeycombed concrete. Experience has shown that certain methods give satisfactory results, and these are illustrated in Figure 11.5. These principles also apply to the handling of graded aggregates and in the transfer of concrete from the mixer to the placement site.

Concrete should fall vertically. In most cases, free fall should be limited [a good rule is 0.9 to 1.5 m (3 to 5 ft)], since segregation is increased as material bounces off form faces and strikes reinforcing steel. However, when formwork is open and concrete drop is unobstructed, free fall to depths of 45 m (145 ft) has proven successful, with no segregation problems. The use of a dropchute or pipe protects concrete during its fall. In deep, narrow forms or in curved forms, concrete placement by pump and hose or by dropchute can be used to advantage. Alternatively, concrete can be initially placed in the lower part of the form through openings specially made for this purpose. In such instances, the concrete should be allowed to flow slowly into the form; it should not enter the form rapidly at an angle from the vertical. When filling walls, concrete placement should be somewhat more advanced at the outer portions of the wall than at the center. This allows any bleed water to flow toward the low point at the center for easy removal as the concrete reaches the top of the form.

On slopes, concrete should be placed and consolidated starting at the bottom, allowing gravity to aid, rather than hinder, the consolidation process. The concrete should be constrained to fall vertically onto the slope; otherwise, coarse aggregate is likely to roll to the bottom. When placing slabs, both horizontal or sloping, concrete



(a) Placement in deep and narrow forms

FIGURE 11.5

Placement of concrete: (a) placement in deep and narrow forms; (b) placement in curved forms; (c) placement on horizontal surfaces; (d) placement on sloping surfaces; (e) consolidation during placement. [Adapted from ACI Committee 304, "Guide for Measuring, Mixing, Transporting, and Placing Concrete (ACI 304R-00)," ACI Manual of Concrete Practice, Part 2. American Concrete Institute, Farmington Hills, MI (2001).]

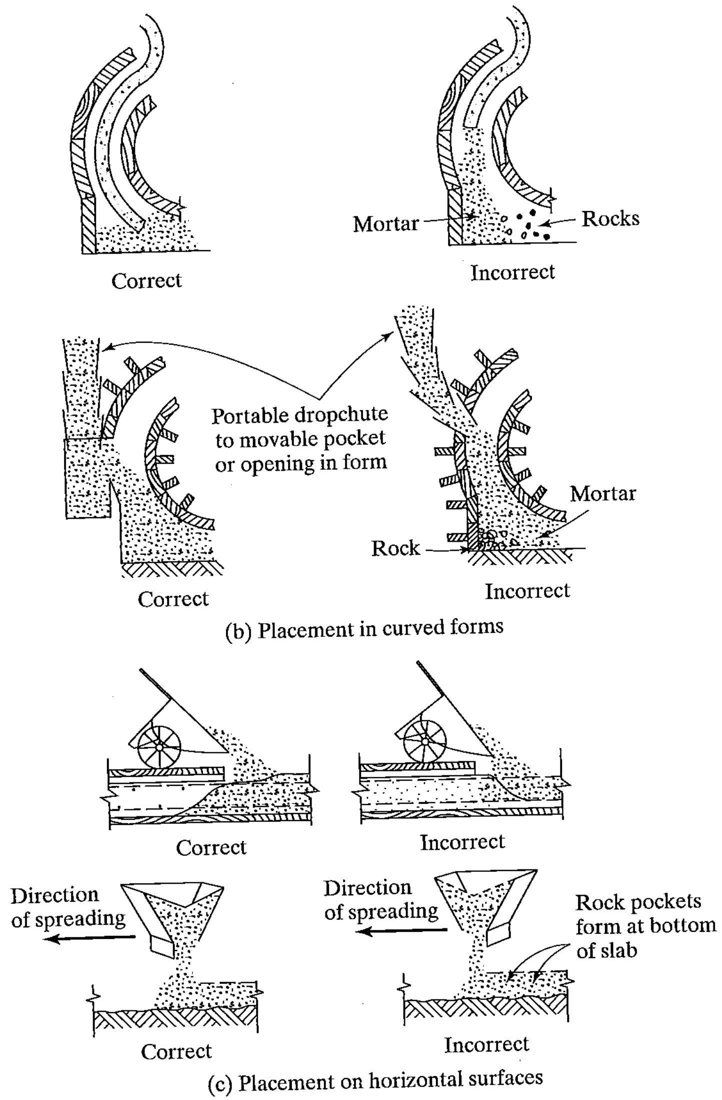


FIGURE 11.5 (b)–(c)

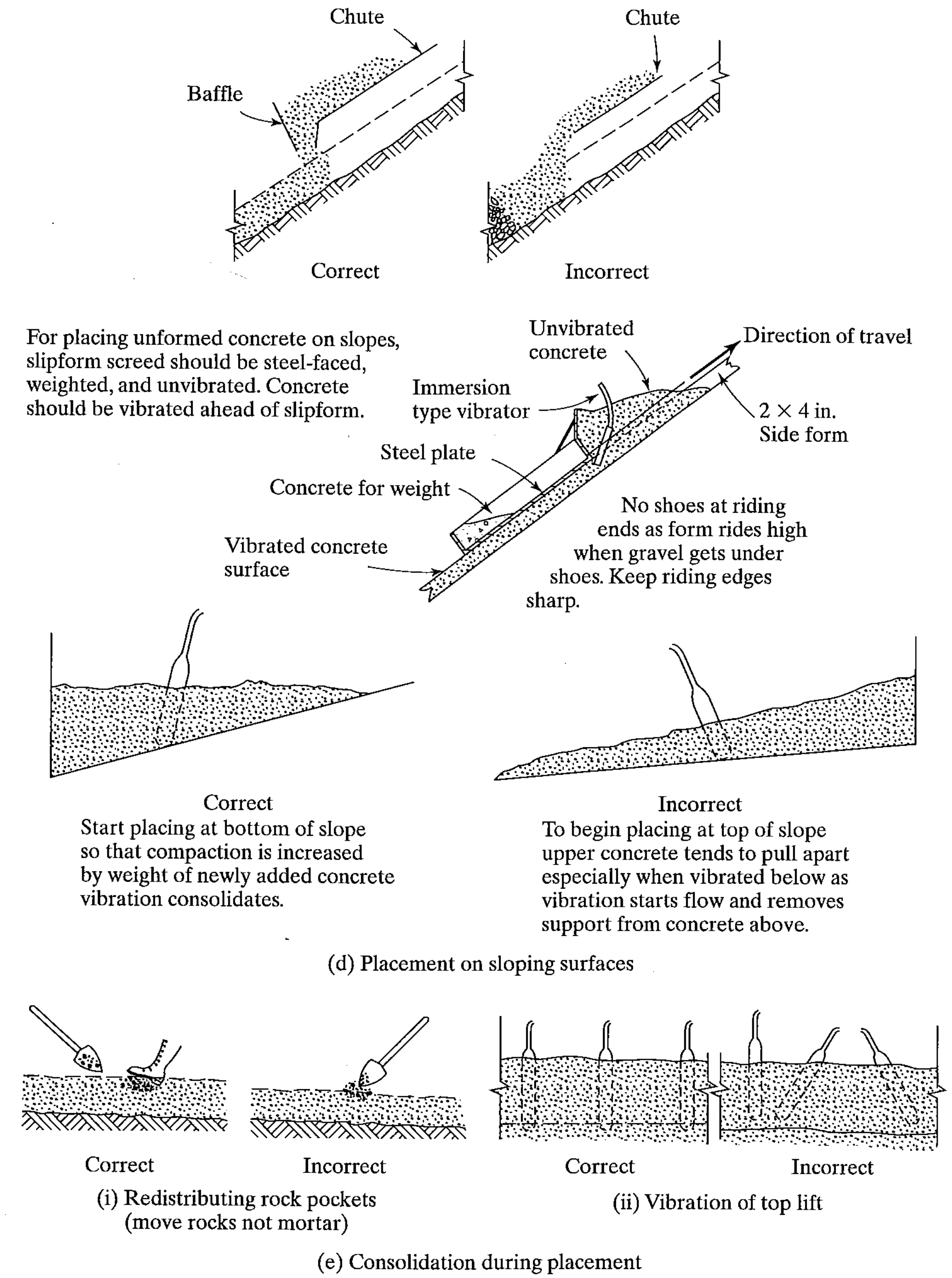


FIGURE 11.5 (d)–(e)

should be dumped into the face of the concrete already placed. Thus, if any segregation has occurred during transportation or occurs during placement, the segregated aggregate can be reworked into the mass. Excessive lateral movement of concrete is not desirable after placement. A strict watch should always be kept for segregation during placement, and action should be taken to correct problems as soon as they arise.

Special Placements

A number of special techniques have been developed for the placement of concrete in certain applications. Those discussed here are slipforming, roller-compacted concrete, preplaced aggregate, shotcreting, the tremie method, and underwater placement.

Slipforming Slipforming involves continuous placement and consolidation of concrete. The method is used for the construction of a variety of structures including silos, chimneys, cooling towers, offshore platforms, storage tanks, monolithic tunnel linings, pavements and high-rise buildings. Virtually all paving operations are now carried out with slipform pavers. The technique is essentially a form of concrete extrusion with self-propelled pavers providing concrete strikeoff, vibration, tamping, and finishing. An auger or paddle distributes the concrete laterally within the paver. Concrete barriers (roadway lane dividers) and curbing are also slipformed.

Vertical slipforming requires formwork to confine the concrete until it has gained sufficient strength to support the concrete placed above and, in many cases, the formwork itself. Forms are moved along the direction of concrete placement at an average rate of 200–300 mm/hr (8–12 in./hr), but rates approaching 600 mm/hr (24 in./hr) have been used. Forms are moved using jacking systems that may be hydraulic, pneumatic, electrical, or manual. Horizontal reinforcing steel for transverse members is accommodated by blocking out a portion of the placement. The blockouts are later removed and the transverse steel inserted. Large concentrations of reinforcing steel should be avoided due to difficulties in concrete placement. For high-rise building construction, ideally the layout should be the same from floor to floor, and wall and column dimensions should be held constant. Reinforcing steel must be tied in to existing reinforcement ahead of form movement so as not to disrupt concrete placement. A spectacular example of a slipformed structure is the CN Tower in Toronto (Figure 11.6), which was erected to a height of 460 m (1500 ft) in about 100 days during the winter months.

Successful slipforming requires careful quality control of materials and production and a properly designed testing program to insure that strength development is adequate. Concrete must possess a high degree of uniformity, and mixes must be cohesive with adequate workability and early strength gain. A concrete slump of 50 to 75 mm (2 to 3 in.) is normally used for vertical placements, while slumps of 25 to 50 mm (1 to 2 in.) are normally used for concrete pavement. Slipforming requires specialized equipment, but the increased productivity and higher quality of the finished concrete can more than offset the higher initial capital investment.

A variant of vertical slipforming is the jumpforming technique. Forms are not moved continuously, but are repositioned (jumped) for the next lift after each casting. Jumpforming thus has similar advantages to slipforming, but may not require as high an investment in specialized equipment.



FIGURE 11.6
View of CN Tower, Toronto.

Roller-Compacted Concrete Roller-compacted concrete (RCC) costs 20 to 30% less than conventional concrete and is used for both mass concrete and concrete pavement construction. For mass concrete, the zero slump material is placed using rock fill and earthmoving equipment (Figure 11.7), while for pavement construction, the concrete is moved using heavy-duty pavers with tamping and vibrating screeds made especially for RCC construction (Figure 11.8). Consolidation of the concrete is external, rather than internal, and is usually accomplished using vibratory rollers. The concrete must have a consistency that allows it to support the rollers and spreading equipment and an aggregate grading and paste content that allows it to be consolidated by rollers. Various degrees of aggregate processing and a range of cement contents are used. For multiple-layer placement, such as used in mass concrete, the materials must be placed so that individual layers can be bonded together.

For mass concrete, the maximum size aggregate is usually limited to 75 mm (3 in.) since larger sizes increase batching costs and increase problems of segregation. The use of 150-mm (6-in.) aggregate for mass concrete, however, can save approximately 15% on the cement required for an equal quality concrete over that required for 75-mm (3-in.) aggregate. Lift thickness should be at least three times the maximum size of the coarse aggregate. For pavement concrete, the maximum size aggregate is usually 19 or 9.5 mm ($\frac{3}{4}$ or $\frac{3}{8}$ in.).

A number of methods are used to proportion roller-compacted concrete. These are based on consistency, trial mixes to produce the greatest economy, and soil-compaction concepts. Coarse aggregates are often blended to obtain a desirable gradation. Proportioning depends on the combined effects of aggregate voids, surface area, and particle shape. Roller-compacted mass concretes have much lower cement contents than conventional concretes [50 to 190 kg/m^3 (80 to 320 lb/yd^3) are typical cement contents], while roller-compacted pavement concretes have cement contents close to those used

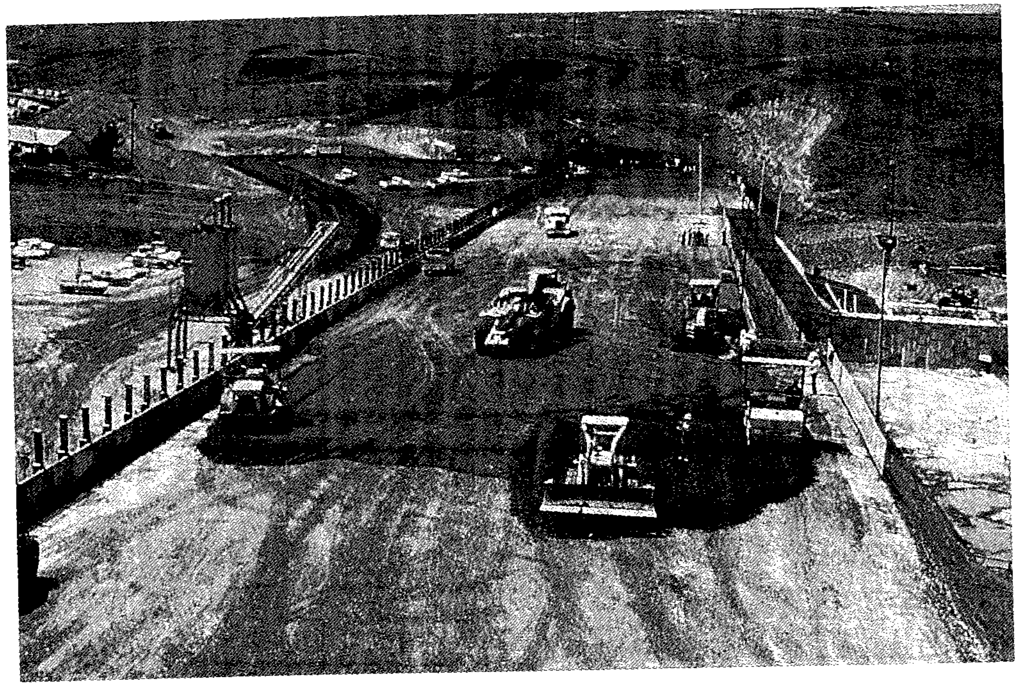


FIGURE 11.7

Roller-compacted mass concrete construction. (Photograph courtesy of U.S. Army Corps of Engineers Waterways Experiment Station.)

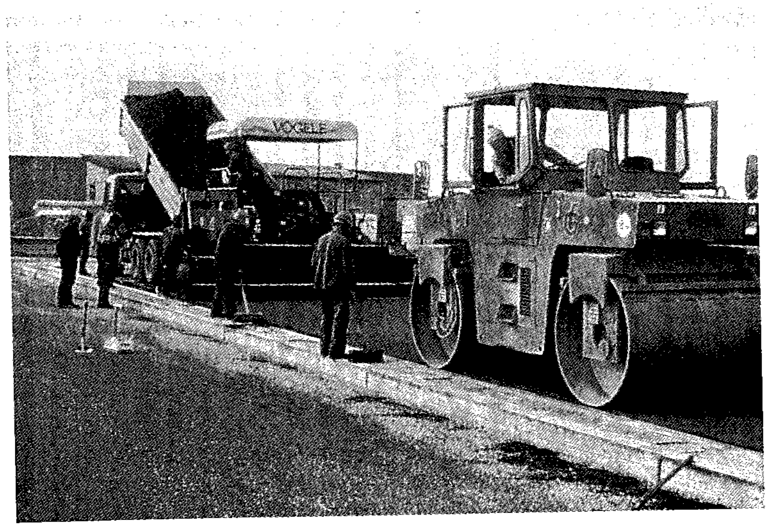


FIGURE 11.8

Roller-compacted concrete paving project. (Photograph courtesy of U.S. Army Corps of Engineers Waterways Experiment Station.)

for conventionally placed concretes. Because of the high aggregate content of roller-compacted concrete, aggregate quality can have a major effect on compressive strength due to the effect of aggregate on water requirements. For the same cement content, a 50% decrease in strength is possible when a low-quality aggregate is substituted for a high-quality aggregate. Roller-compacted concrete has demonstrated extremely good resistance to water erosion and appears to provide excellent freeze–thaw durability except under very severe (saturated) exposure conditions.

While RCC pavements can usually be placed in a single lift, mass construction involves the placement of multiple lifts over large areas. Because construction should be nearly continuous and because construction rates are much higher than with conventional concrete, large aggregate stock piles are required before construction begins when multiple layers must be placed, such as in dam construction. In North American practice, the most common compacted lift thickness for mass RCC is about 300 mm (12 in.). However, in Japan compacted lift thicknesses of 0.5 to 1.0 m (1.6 to 3.2 ft) have been used. Each lift is built up in several layers before compaction begins, as the concrete is transported and deposited at the job site. Conveyor belts and earth hauling equipment are often used. The concrete is spread in several layers using track-mounted equipment, such as bulldozers, until the full uncompacted lift thickness is attained. The concrete is then consolidated using vibratory rollers. Compaction should be completed as soon as possible after the concrete has been spread, with 10 min being the normal maximum limit. Total time from initial mixing to consolidation should be limited to 40 min. If too much time has elapsed or if the concrete has been damaged, it may still appear satisfactory after rolling. However, unlike properly consolidated roller-compacted concrete, inadequately consolidated concrete is easily displaced using an air hose. Thus, high-pressure air provides an effective method to both test and remove the material for replacement with properly compacted concrete.

Horizontal joints, such as may occur in mass concrete when construction is stopped for a period of time, must be watertight. This is normally accomplished using a higher workability material for the initial lift. This lift may be a thin layer of bedding mortar or a highly sanded conventional concrete. If the initial lift proves to be too fluid, this can be handled by spreading the concrete over a wider area and using a thinner lift before placing the stiff roller-compacted concrete. Roller-compacted concrete can be constructed using forms and precast concrete to provide smooth facings. Careful control of consolidation must be exercised near the edge of a placement to ensure that the consolidation equipment is properly supported and that any forms or facing members are not damaged. This may require the use of smaller compaction equipment.

Although all concrete depends on the control exercised during construction, roller-compacted concrete is especially dependent on job site control because of the high rates of placement that are normally attained with the material. Test cylinders may represent only an historical record, since low-quality concrete may be covered by many meters of material before the low strength is observed. A particular area of emphasis should be the cement content during placement. Roller-compacted concrete that is low on cement will not look particularly different from the normal lean mix. It will handle and compact in a similar manner, but its strength will be very low.

As with conventional concrete, roller-compacted concrete must be cured and protected from the weather. Light rain will not cause a problem, as long as vehicular

traffic does not work the moisture into the surface or deposit mud from adjacent haul roads. Construction can continue as soon as the concrete surface returns to a saturated surface dry condition. Compacted lifts should be kept in a damp condition without ponding water. Covering with plastic or providing a light spray of water that does not erode the surface have proven to be effective curing methods. The final lift should be cured for at least 14 days. Curing compound is not recommended because of the difficulty in obtaining full coverage on the rough surface and because of the advantages inherent in using moist curing.

Preplaced Aggregate This method entails packing forms with well-graded coarse aggregate and injecting structural mortar (or grout) into the mass to fill the voids. This method is adaptable to underwater concreting, for repairs of existing structures, or for situations where conventional placements are difficult, such as heavily reinforced sections used for seismic or other applications. Because the aggregate can be packed more densely than in ordinary concrete, less cement paste need be used and concrete of low volume change results. However, this method requires skill and experience to ensure complete filling of the void space. The secret of good placement is the use of a high-quality, well-graded, clean aggregate conforming to ASTM C 33 (except for gradation), and a cohesive, yet fluid, mortar that develops good strength. The coarse aggregate is graded to obtain as low a void content as possible, and the sand used in the mortar is much finer than normal. Any type of portland cement may be used, but admixtures are generally required to attain the required grout properties. Aluminum powder is usually added to the grout to cause it to expand and fill the voids between the coarse aggregate particles (Chapter 20). Concrete made by the preplaced aggregate method will have direct contact between aggregate particles, whereas this does not occur in concrete made by conventional means. This may affect the elastic properties of the concrete, as well as its mode of fracture.

Shotcreting Concrete can be applied pneumatically by spraying it from a nozzle by means of compressed air. This process is known as *shotcreting* or *guniting*. The latter term is in more common usage in Europe, but remains the registered trademark of the Allentown Pneumatic Gun Company, which first developed the process on a commercial basis early in this century. Shotcrete is applied using either the dry-mix or the wet-mix process. In the dry-mix process, the solid materials are blended prior to spraying and the water is added just before ejection at the nozzle. The amount of water is adjusted by the nozzleman; it requires considerable experience to get a mix that is neither too wet nor too dry. Schematic views of a dry-mix gun and nozzle are shown in Figure 11.9. In the wet-mix process, all of the ingredients, including water, are mixed prior to entry into the delivery equipment. Compressed air in the nozzle drives the material toward the receiving surface. Wet-mix shotcrete is often delivered by a ready-mix truck. A schematic of a wet-mix nozzle is shown in Figure 11.10. The quality of wet-mix shotcrete is much less dependent on the nozzleman than it is for dry-mix shotcrete. Concrete for shotcreting is generally made with fairly fine aggregate: pea gravel (10-mm maximum size) and sand, although aggregates up to 20-mm maximum size can be used. Fiber-reinforced concrete can also be shotcreted.

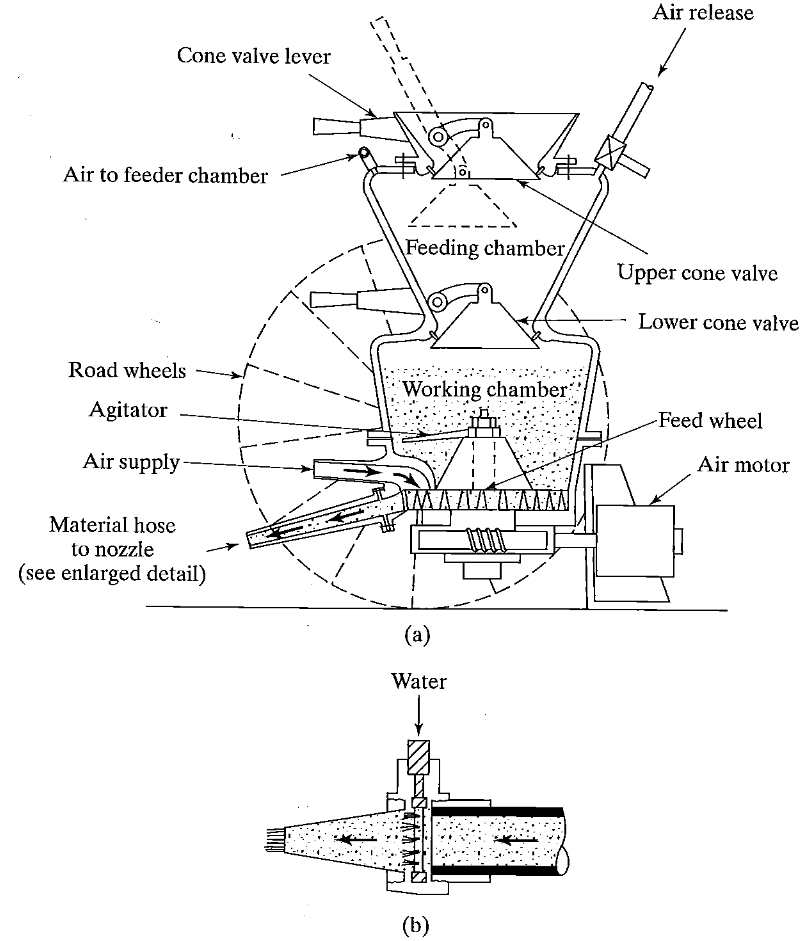


FIGURE 11.9

Schematic view of dry-mix shotcreting equipment: (a) supply chamber; (b) nozzle. [From J. R. Illingworth, *Movement and Distribution of Concrete*, McGraw–Hill Co. (U.K.) Ltd., (1972). Reproduced by permission of The Cement Gun Co. Ltd.]

The concrete is ejected from the nozzle at high velocity so that the concrete is well compacted at the surface. Thus, a dense concrete with a low *w/c* ratio can result with compressive strengths of 40 to 55 MPa (6000 to 8000 lb/in.^{2}) at 28 days, and it forms a good bond to steel and substrate. Wet-mix shotcrete generally has a higher *w/c* ratio and a greater similarity to cast-in-place concrete than does dry-mix shotcrete. For both processes, about $\frac{1}{4}$ to $\frac{1}{2}$ of the material rebounds on impact and is not retained on the surface. Most of this is the coarser material, so the in-place shotcrete is richer than the original mix. Admixtures are often used in shotcreting, the most common being quick-setting admixtures that are claimed to reduce rebound and give very rapid strength development in a matter of hours. Pozzolans can be incorporated in both dry

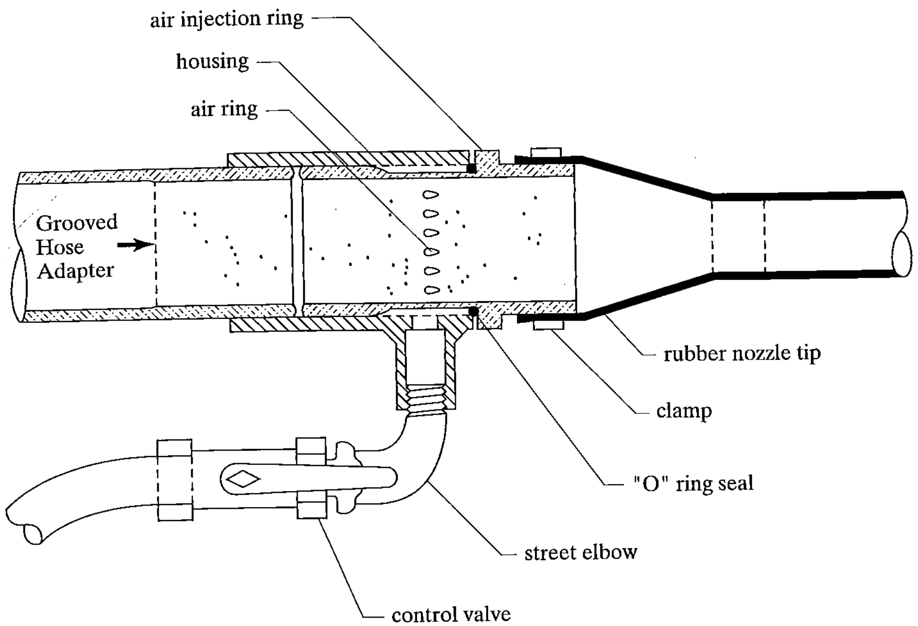


FIGURE 11.10

Wet-mix shotcrete nozzle. [From ACI Committee 506, "Guide to Shotcrete (ACI 506R-90)," *ACI Manual of Concrete Practice,* Part 5. American Concrete Institute, Farmington Hills, MI (2001).]

and wet-mix shotcrete, producing a material with greatly increased cohesiveness that allows thicker layers to be built up. When silica fume is used for dry-mix shotcrete, best results are obtained when it is preblended in the undensified form. As with conventional concrete, superplasticizers are needed when silica fume is used in wet-mix shotcrete.

Shotcreting is an ideal way to place concrete on vertical or steeply sloping surfaces. It is built up in layers by repeated passes across the face and reinforcing steel may be incorporated in the concrete. It has been used to construct tunnel linings, stabilize rock faces, and provide surface supports, without the need for formwork, and can be used in any situation where conventional formwork would be difficult to use. But its most versatile use is perhaps in the repair and restoration of existing structures. Shotcreting can restore structural integrity to damaged members or provide new, protective coatings over such diverse materials as brick and timber. Prior to the shotcreting operation, most surfaces require some preparation, including removal of any damaged material. The surface of both structural and reinforcing steels should be blast-cleaned to achieve a good bond.

Tremie Concrete The tremie method of placement is used for pouring concrete under water. The concrete is delivered by gravity through a rigid vertical pipe (the tremie) fed through a funnel-shaped hopper (Figure 11.11). The tremie method is designed to minimize the occurrence of entrapped air and other voids, and it has the

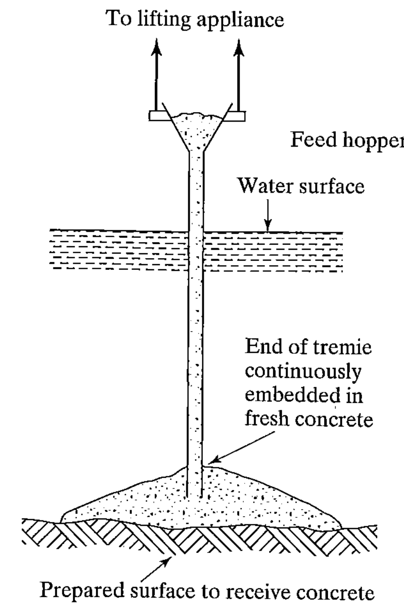


FIGURE 11.11

Use of the tremie for underwater concrete placement. [From J. R. Illingworth, *Movement and Distribution of Concrete,* McGraw–Hill Co. (U.K.) Ltd. (1972). Reproduced by permission.]

advantage of causing minimal surface disturbance, which is particularly important when a concrete–water interface exists. For successful tremie placement, a steady, uninterrupted flow of concrete through the tremie pipe should be maintained and the seal with the placed concrete kept at all times. Blockages in the pipe can result from even short delays and will require the tremie to be withdrawn and placement started again. At the beginning of the placement, the tremie is sealed with a plug, called a go-devil or pig, that moves downward as placement begins so that the concrete is not disturbed as it displaces the water in the pipe. Alternatively, the bottom of the pipe is sealed with an endplate, and the pipe is dewatered prior to concrete placement. Once placement begins, the tremie should be embedded 1 to 1.5 m (3 to 5 ft) in the fresh concrete. If the seal is broken or if the tremie must be moved horizontally, placement must be stopped, the tremie resealed at the bottom, and placement restarted with the tremie well embedded in the fresh concrete. A go-devil should not be used in this case, because water displaced from the tremie as placement is restarted will wash or scour the concrete.

Because the concrete must flow into place, it should be designed to have a high slump [150 to 250 mm (6 to 10 in.)] without a tendency to segregate or bleed. To obtain flowing, cohesive concrete, it is necessary to use a high percentage of sand: 45 to 55% by volume of the total aggregate. Consequently, mixes will have high cement contents. Water-reducing admixtures, air-entraining agents, pozzolans and antiwashout admixtures can all be used to advantage in designing tremie concrete. A maximum aggregate size of 19 mm ($\frac{3}{4}$ in.) is recommended for reinforced concrete and 38 mm ($1\frac{1}{2}$ in.) for nonreinforced concrete. Pipe diameters in the range of 200 to 300 mm (8 to 12 in.) can handle concrete with these aggregate sizes.

Because of its high workability, tremie concrete can move laterally as much as 20 m (70 ft). However, a pipe spacing of 4.5 m (15 ft) is usually recommended as optimum. If

a pipe must be moved laterally, it should not be moved through the plastic concrete. Rather, placement should be stopped, the pipe withdrawn, resealed, and reinserted at the new location, and placement restarted.

Underwater Placement The tremie method is only one method of placing concrete under water, although the considerations applying to tremie placement also apply to other methods. Pumps can be used in place of a tremie for underwater placement, or the preplaced aggregate method can be used. Special bottom-dump buckets have been designed for underwater placement (Figure 11.12). Such a bucket has "skirts," which are lowered while the concrete is discharged to protect it from the surrounding water. Other methods include the use of special fabric forms that are specialized for underwater placement and construction of diaphragm or slurry walls in which concrete is placed within a trench while the upper surface is under water.

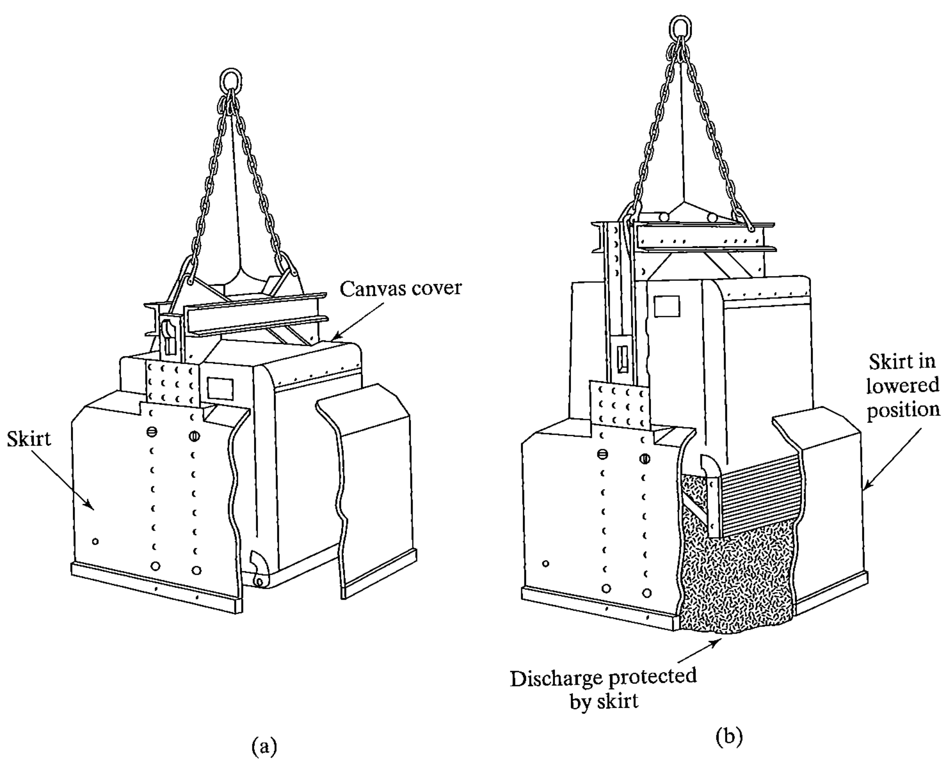


FIGURE 11.12

Placement with the bottom dump bucket; (a) filled; (b) discharging. [From J. R. Illingworth, *Movement and Distribution of Concrete,* McGraw-Hill Co. (U.K.) Ltd. (1972). Reproduced by permission of John Grist Ltd.]

Consolidation

After placement, the concrete should be worked to eliminate voids and entrapped air and to consolidate the concrete into the corners of the forms and around the reinforcing steel. Most concrete is consolidated by vibration. Proper vibration allows stiffer mixes to be used and generally leads to better consolidation and a superior finish. Concrete slumps may be as little as one-third of those consolidated by hand, but all concretes (even high-slump, superplasticized concretes) can benefit from vibration, if the equipment and duration of vibration are selected to match the properties of the concrete. Vibration is needed for proper compaction of concrete with less than a 50-mm (2-in.) slump. Overvibration brings excess paste to the surface, enhances bleeding, and causes loss of entrained air (Figure 11.13).

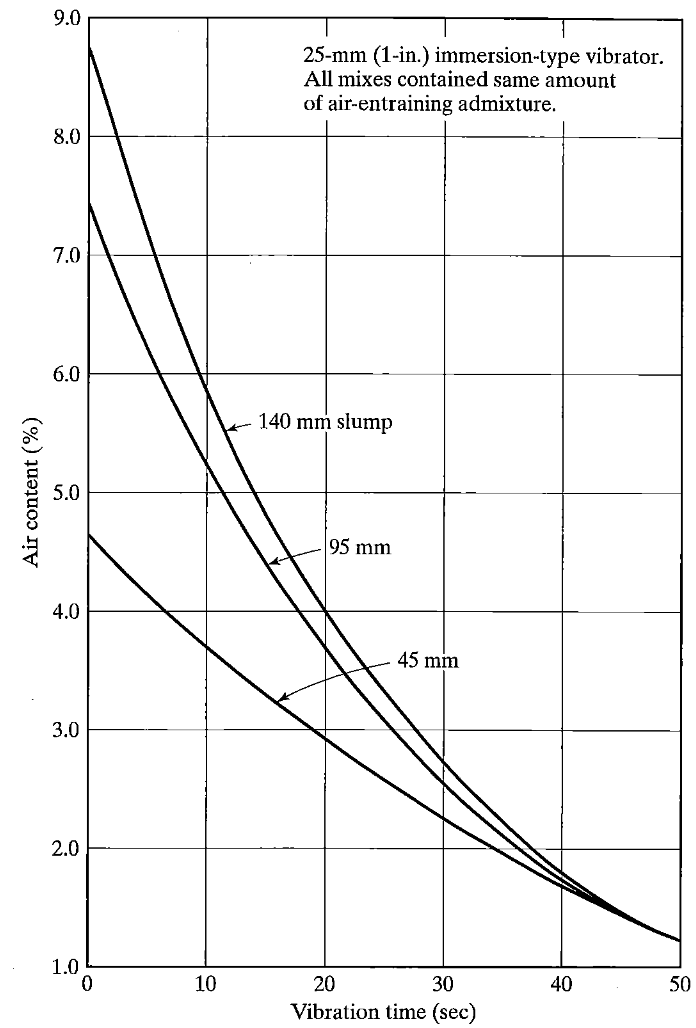


FIGURE 11.13

Relationship between slump, duration of vibration, and air content. [From R. S. Brewster, *Effect of Vibration Time upon Loss of Entrained Air from Concrete Mixes,* Materials Lab. Rep. No. C-461, Research and Geology Division, Bureau of Reclamation, Denver, Nov. 25 (1949). Cited in *Design of Concrete Mixes,* 6th Canadian ed., Canadian Portland Cement Association, Ottawa, Ontario (1995).]

Concrete Vibrators Vibrators apply periodic forces to the concrete with an eccentric rotating mass. These forces first cause the concrete to subside and level out and then force entrapped air out of the concrete. The concrete flows (or "liquefies") under the shear forces accompanying the vibration, and the concrete is compacted away from the vibrator. Internal or immersion vibrators (often called spud or poker vibrators) operate at frequencies in the range 5500 to 15000 rev/min (90 to 250 Hz), while external vibrators are generally run at 1000 to 12000 rev/min (16 to 200 Hz). The response of concrete depends not only on its workability, but also on the frequency and amplitude of vibration.

External vibrators can be clamped to formwork, when the proper use of internal vibrators is not possible (congested reinforcement, narrow spaces, curved sections, or in slipforming). Vibrating screeds or pan-type vibrators are commonly used on flatwork, especially slipformed paving. Consolidation on vibrating tables is used in precast work. External vibrators require more power as they impart energy to the formwork also and are generally less effective than internal vibrators. Forms must be strong and rigid and remain watertight, and for this reason, metal forms are best used with the external vibrators.

Internal vibrators are most suitable in general construction. The energy imparted by the head of the vibrator excites the solid particles in the concrete mix, causing it to flow. However, the concrete does not move uniformly, as can be seen in Figure 11.14. The coarse aggregate particles are propelled from the vibrator head preferentially because of their greater mass. Momentum is transferred through particle collisions. Mortar then begins to flow between the coarse aggregate. Undervibration leaves excessive amounts of entrapped air and is a frequent cause of honeycombing, since the coarse aggregate has been excited, but the mortar has not had sufficient time to flow to the same extent.

Internal vibrators should be selected based on the characteristics of the concrete. Generally speaking, stiff concrete with a large maximum-size coarse aggregate requires a larger vibrator with a higher amplitude and a lower frequency than a more fluid concrete or a concrete with smaller coarse aggregate. Table 11.3 provides guidance on vibrator selection.

For efficient consolidation, the vibrator should be allowed to sink into concrete under its own weight and should penetrate any underlying layer of fresh concrete by at least 150 mm (6 in.). The rapid penetration tends to force the concrete upward and outward, thereby helping air to escape initially. The vibrator should be held stationary until the coarse aggregate particles settle below the surface, a layer of mortar rises to the top, and large air bubbles no longer escape from the concrete within the "radius of action" of the vibrator. Vibration is complete when cement paste begins to appear around the vibrator. The vibrator should then be withdrawn slowly to allow the concrete to close in around the vibrator head as it is removed. As an alternate method, the vibrator can be moved up and down in the concrete after penetration and during removal. The time required for proper consolidation varies with the vibrator and the concrete, but typically ranges from 5 to 20s. Overvibration will cause undesirable segregation of the mix; high-slump mixes with high mortar contents are especially susceptible. However, undervibration is much more common than overvibration.

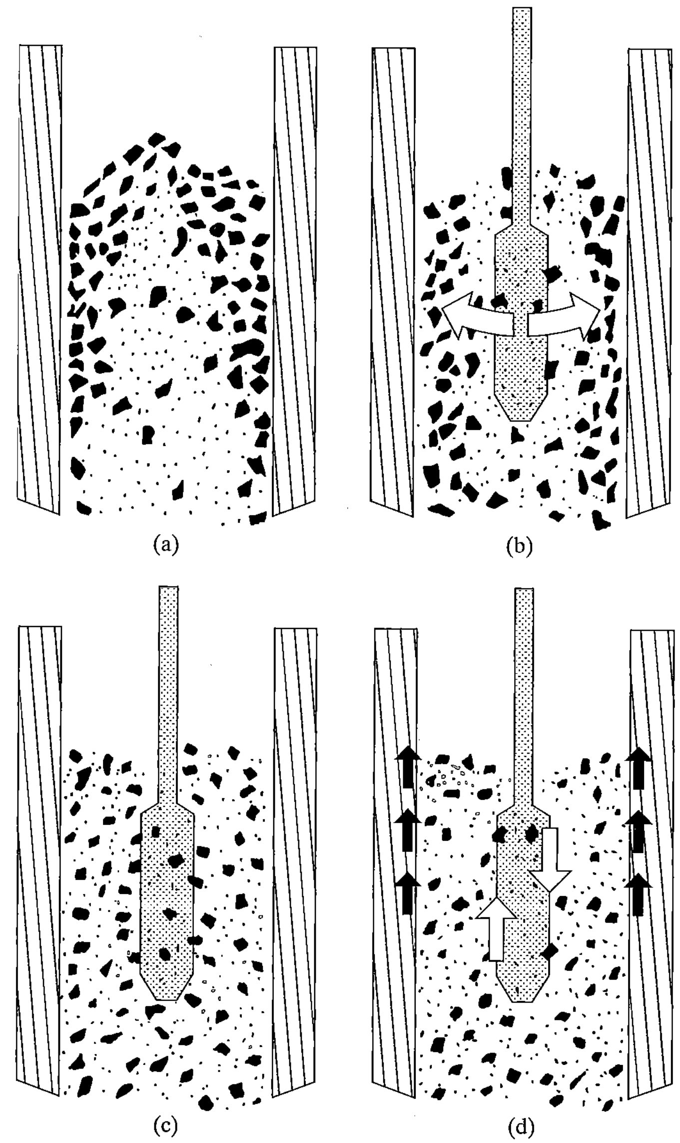


FIGURE 11.14

Idealized representation of the influence of a high-frequency vibrator on concrete consolidation: (a) The mix is introduced into the form; (b) the vibrator moves aggregate closer together at the form face and cement—sand mortar begins to move outward, and air pockets collect on the faces of forms; (c) the mortar continues to move through the coarse aggregate toward the face of the form; (d) the movement of mortar toward the face is complete. As the operator moves the vibrator down and up, air bubbles move upward along the form face and out of the concrete. [From *Concrete Construction*, Vol. 17, No. 11, Concrete Construction Publications, Inc. pp. 536–538 (1972). Produced herein with permission. Copyrighted 2001 by Hanley–Wood, LLC, 426 South Westgate St., Addison, IL 60101.]

TABLE 11.3 Range of Characteristics, Performance, and Applications of Internal Vibrators[a]

Column 1	*2*	*3*	*4*	*5*	*6*	*7*
				Approximate values of		
Group	*Diameter of head, mm (in.)*	*Recommended frequency, vibrations per min (H_z)*	*Average amplitude, mm (in.)*	*Radius of action, mm (in.)*	*Rate of concrete placement, m³/hr per vibrator (yd³/hr)*	*Applications*
1	20–40 (0.75–1.5)	9000–15,000 (150–250)	0.4–0.8 (0.015–0.03)	80–150 (3–6)	0.8–4 (1–5)	Plastic and flowing concrete in very thin members and confined places. May be used to supplement larger vibrators, especially in prestressed work where cables and ducts cause congestion in forms. Also used for fabricating laboratory test specimens.
2	30–60 (1.25–2.5)	8500–12,500 (140–210)	0.5–1.0 (0.02–0.04)	130–250 (5–10)	2.3–8 (3–10)	Plastic concrete in thin walls, columns, beams, precast piles, thin slabs, and along construction joints. May be used to supplement larger vibrators in confined areas.
3	50–90 (2–3.5)	8000–12,000 (130–200)	0.6–1.3 (0.025–0.05)	180–360 (7–14)	4.5–15 (6–20)	Stiff plastic concrete [less than 80-mm (3-in.) slump] in general construction such as wall, columns, beams, prestressed piles, and heavy slabs. Auxiliary vibration adjacent to forms of mass concrete and pavements. May be gang mounted to provide full-width internal vibration of pavement slabs.
4	80–150 (3–6)	7000–10,500 (120–180)	0.8–1.5 (0.03–0.06)	300–500 (12–20)	11–31 (15–40)	Mass and structural concrete of 0 to 50-mm (2-in.) slump deposited in quantities up to 3 m³ (4 yd³) in relatively open forms of heavy construction (power-houses, heavy bridge piers, and foundations). Also auxiliary vibration in dam construction near forms and around embedded items and reinforcing steel.
5	130–180 (5–7)	5500–8500 (90–140)	1.0–2.0 (0.04–0.08)	400–600 (16–24)	19–38 (25–50)	Mass concrete in gravity dams, large piers, massive walls, etc. Two or more vibrators will be required to operate simultaneously to melt down and consolidate quantities of concrete of 3 m³ (4 yd³) or more deposited at one time in the form.

Column 3—While vibrator is operating in concrete.
Column 4—Peak amplitude (half the peak-to-peak value), operating in air.
Column 5—Distance over which concrete is fully consolidated.
Column 6—Assumes insertion spacing is $1\frac{1}{2}$ times the radius of action, and that vibrator operates two-thirds of the time concrete is being placed.
Column 5 and 6—These ranges reflect not only the capability of the vibrator, but also differences in workability of the mix, degree of deaeration desired, and other construction.
[a]Adapted from ACI Committee 309, "Guide for Consolidation of Concrete (ACI 309R-96)," *ACI Manual of Concrete Practice,* Part 2, American Concrete Institute, Farmington Hills, MI (2001).

The vibrator should be inserted vertically, at regular spacings, as shown in Figure 11.5e, so that the regions of concrete affected by the vibrator clearly overlap. Spacings of about $1\frac{1}{2}$ times the radius of action [typically 300–450 mm (12–18 in.)] are usually satisfactory. The radius of action is usually smaller near reinforcing steel than in plain concrete. In thin slabs, the vibrator should be tilted so that the vibrator head is fully embedded in the concrete. However, for slabs on the ground, care should be taken to limit vibrator contact with the subgrade since this may result in contamination of the concrete. For reinforced structures, the vibrator may come in contact with the reinforcement without damaging the concrete or the bond between the steel and the concrete, as long as good vibrator technique is followed. The vibrator should not touch the bottom of the form if the slab is used in *tilt-up* construction (the slab will become a wall) and the bottom will serve as an architectural surface. Vibrators should not be used to move concrete laterally in forms, as this causes segregation.

Revibration of concrete an hour or two after initial consolidation may be needed to weld successive lifts together. This process can be beneficial and can be used to improve concrete consolidation by eliminating any cracking, voids, or weak areas created by subsequent settlement or bleeding, particularly around reinforcing steel or other embedded materials, or loss of moisture by defects in formwork. Revibration also improves steel–concrete bond strength for top-cast bars in high-slump concrete and is generally most effective for the upper portions of a concrete placement for concretes placed with a slump of 75 mm (3 in.) or more, when used prior to initial set. However, revibration is not universally beneficial, since it has been shown to reduce the bond strength of reinforcing bars that have not been affected by settlement or bleeding, that is, top-cast bars in low-slump concrete and bottom-cast bars in both low-slump and high-slump concrete.

Vacuum Dewatering Vacuum dewatering (or vacuum processing) is a method of consolidation that involves the application of a vacuum to a fresh concrete surface. The method is especially useful for the consolidation of horizontal surfaces that will be subject to considerable wear. For slabs, a rubber mat is laid on the surface (Figure 11.15) to provide a seal, and a vacuum applied inside the mat sucks out water from the concrete. Filter pads prevent fine particles from being removed with the water. In this way, 15 to 25% of the original water can be removed from the upper 150 to 300 mm (6 to 12 in.) of a slab (depending on the length of the treatment). As much as 40% can be removed from the concrete near the surface. The lower *w/c* ratio and the effective compaction of the concrete give dramatic improvements in strength and durability (see Figure 11.16). Improvements in performance are greater than if calculated simply on the basis of changes in overall *w/c* ratio because of the tendency for the *w/c* ratio to be higher at the surface of a slab due to bleeding. While the process can be used for any concrete, best results are obtained for concretes with low fines (including low cement content), low sand contents, and coarse sands.

Vacuum dewatering is started after the concrete has been vibrated and screeded. Subsequently, regular finishing can begin immediately so that dewatering can actually speed up the finishing process. Power finishing is needed because of the hardness of the surface. Initially, a planing disc should be used to remove slight irregularities caused by the dewatering process, followed by final troweling.

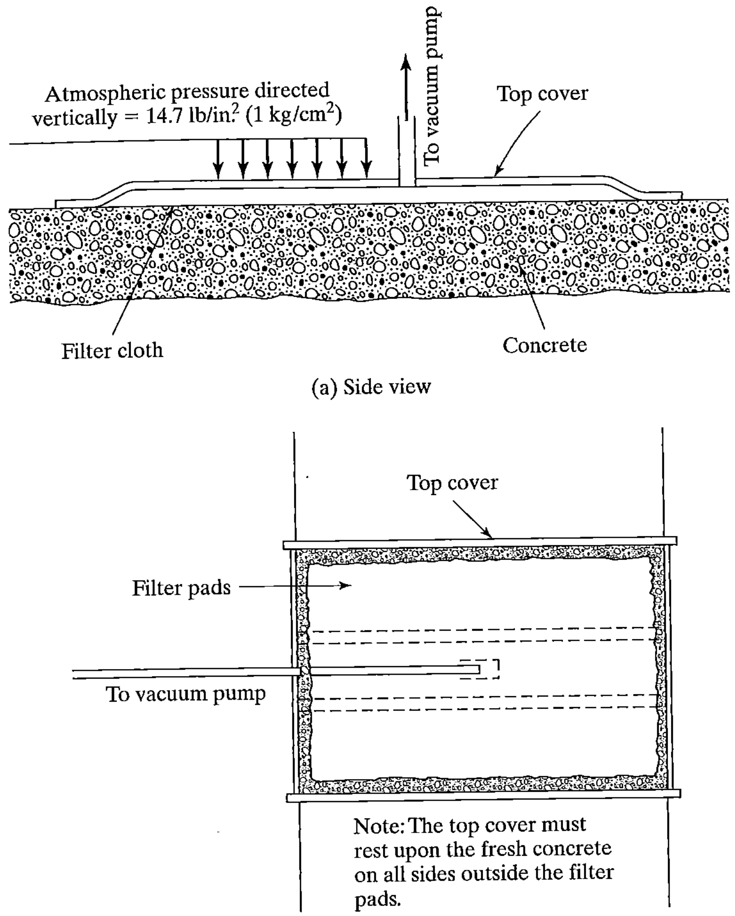


FIGURE 11.15

Schematic representation of vacuum dewatering; (a) side view; (b) top view. [From H. Wenander, *Concrete Construction*, Vol. 20, No. 2, Concrete Construction Publications, Inc, pp. 40–42 (1975). Produced herein with permission. Copyrighted 2001 by Hanley-Wood, LLC, 426 South Westgate St., Addison, IL 60101.]

Vacuum processing can also be applied to inclined and vertical concrete surfaces, providing reduced concrete pressures on formwork and early form removal. In addition to increased strength and durability, the appearance of the concrete is improved due to removal of surface air voids along with the water.

Casting in Lifts

It is desirable to cast concrete in monolithic units between construction joints where possible. Deep placements should be cast in successive horizontal layers, or lifts, avoiding

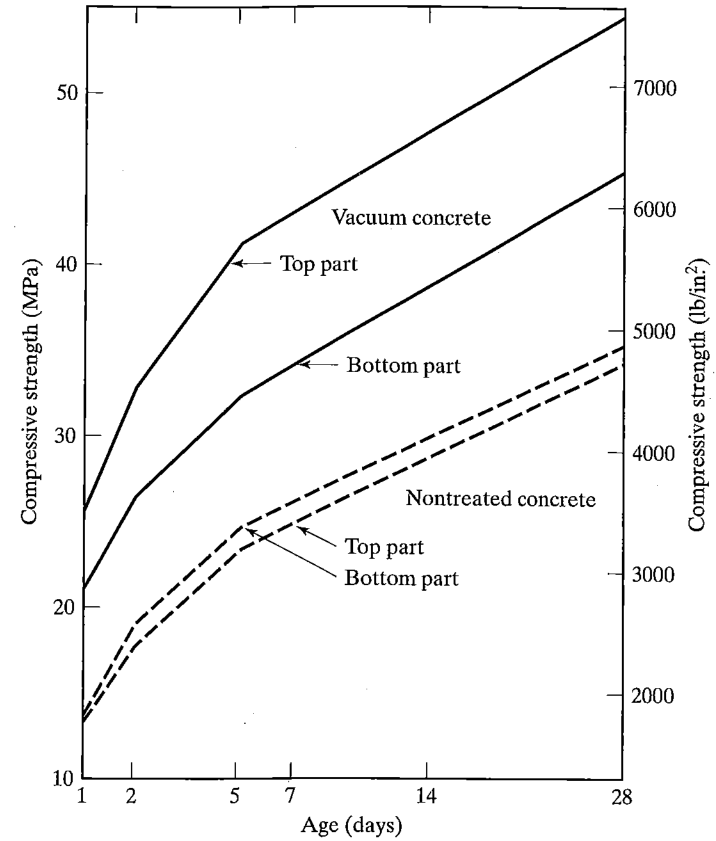


FIGURE 11.16

Effect of vacuum dewatering on the strength of concrete. [From H. Wenander, *Concrete Construction*, Vol. 20, No. 2, Concrete Construction Publications, Inc, pp. 40–42 (1975). Produced herein with permission. Copyrighted 2001 by Hanley-Wood, LLC, 426 South Westgate St., Addison, IL 60101.]

sloping surfaces as much as possible. Adjacent lifts should be "knitted" together by ensuring that the internal vibrator passes 150 mm (6 in.) into the lower lift. Successive lifts should not be more than about 500 mm (20 in.) thick and should only be several cm (a few inches) thick if external vibration is used. If excessive vibration of the lower lift has not occurred, there should be no discontinuity between lifts. The slump of concrete should be less in the upper lifts, for the tendency of even well-designed concrete to bleed slightly through settlement will increase the water content in the upper levels.

When the concrete in a lift hardens before the next lift can be placed, a *cold joint* or *construction joint* will result, since the two layers can no longer be consolidated together. The inherent weaknesses of such a joint can be minimized by proper procedures. The surface of the last lift should be left in a roughened state to provide a good mechanical bond. Before placement of the next lift, the surface should be scarified to remove any laitance (see Section 9.1). Air–water jets, wire brooming, or even sandblasting are useful techniques. The concrete should be dampened and a layer of mortar worked well into the surface. This ensures a good bond between fresh and hardened concrete and reduces the possibility of entrapped air or honeycombing at the bottom of the upper lift.

11.4 FINISHING CONCRETE

Finishing Flatwork

Special techniques have been developed for finishing the surfaces of concrete slabs for floors and pavements. The object is to produce a dense, compacted, properly graded surface suitable for the conditions of service. Proper finishing of good-quality concrete can ensure a maintenance-free surface, but cannot offset the inherent deficiencies of poorly designed concrete. The sequence of finishing operations is as follows:

Screeding Excess concrete is struck off to bring the surface to the proper grade. This can be done mechanically or by hand. The screed is moved back and forth across the concrete with a forward motion, and surplus concrete is left in front to fill in any low spots. Vibrating screeds can aid consolidation as well as strike-off the concrete.

Bullfloating Immediately after screeding, before bleed water has risen to the surface, the bullfloat is used to embed large aggregate particles and remove any remaining bumps and hollows. This long-handled tool is used on relatively large placements by drawing the trailing edge of the float across the slab. For smaller slabs or in tight areas, the same process (now called *darbying*) is accomplished with a darby, which is swept in a circular motion across the slab surface.

Straightedging For slabs that must meet high standards for flatness and levelness, a highway-type straightedge [1.8–3.6 m (6–12 ft) long] is used in place of the bullfloat immediately after screeding in a cutting and filling operation. The process reveals irregularities left during the screeding operation and must be completed before excess bleed water rises to the surface.

Floating The concrete is allowed to harden until all bleed water has disappeared and the concrete can be walked on without leaving an indentation greater than 5 mm ($\frac{1}{4}$ in.). The surface is then floated with flat wood or metal blades. This process, which can be done by hand or by machine, firmly embeds the aggregate, compacts the surface, and removes any slight imperfections that may remain. Floating tends to bring cement and water to the surface, thus floating too early, or for too long, can be damaging and weaken the surface by forming a layer of high *w/c* ratio paste.

Troweling When floating is finished, the surface may be steel-troweled if a really smooth, dense, wear-resistant surface is required. Like floating, troweling can be accomplished by hand or by machine. At this stage, the object is compaction and surface defects cannot be repaired. The concrete is ready for troweling when no paste accumulates on the blade and the trowel gives a distinctive ringing sound when drawn across the surface. Again, troweling too early can be damaging, and troweling with the blade at too steep an angle can result in blistering of the surface. More than one troweling can be done if desired.

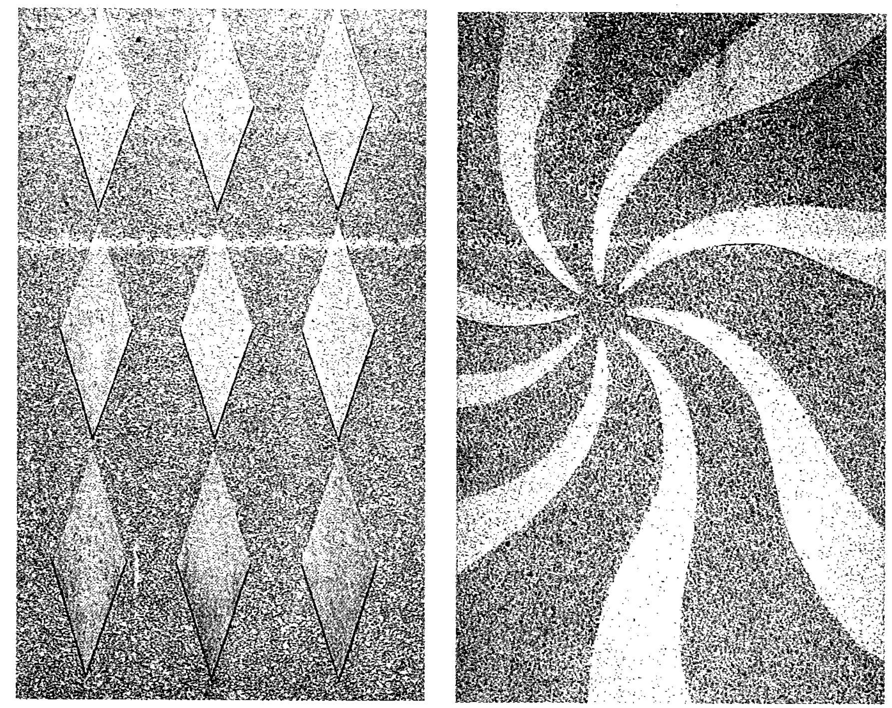
Texturing A well-troweled surface will be smooth and shiny and prone to slipping or skidding, especially when wet. A textured surface may be needed for skid resistance. Scoring the surface with a wire broom, coarse fiber broom, artificial turf, or metal tines is common. Drawing burlap across the surface will also provide texture. These texturing operations are carried out after bullfloating/straightedging and, when used, represent the final finishing process.

Special wear-resistant aggregate toppings are often applied by distributing aggregate evenly over the top of freshly cast concrete, firmly embedding it with a darby, followed by floating and troweling. Decorative aggregates for exposed finishes may also be applied in this fashion. After floating, excess paste at the surface can be removed by gentle washing and brushing to expose the aggregate.

Hardening When the durability or wear resistance is suspect, these properties can be improved by treating the concrete surface with certain chemicals. The treatment will cause the precipitation of insoluble compounds in the pores close to the surface, thereby making a stronger, denser layer. Formulations based on sodium silicate or on fluorosilicates are the most common. Reaction with the calcium hydroxide in the paste forms additional C–S–H. ACI Committee 302, Construction of Concrete Floors, recommends the use of these materials only as an emergency measure to correct deficiencies.

Mechanical Finishes

Mechanical finishes are applied once concrete has hardened and can be used on both vertical and horizontal surfaces. The technique is often used for architectural concrete applications. Mechanical finishes include abrasive action (sandblasting and grinding) and fracturing processes (tooling). Sandblasting can be used for exposed aggregate finishes due to the preferential abrading of the hardened paste. Light sandblasting may give a slightly roughened surface without exposing aggregate (Figure 11.17a) or may



(a) (b)

FIGURE 11.17

Examples of decorative effects by selective sandblasting. (Photographs courtesy of Portland Cement Association.)

be used to expose aggregates to a depth of as much as 12 mm ($\frac{1}{2}$ in.) (Figure 11.17b). The depth of abrasion depends on the size of grit, age of concrete, and time of treatment. The use of rubber templates to protect certain areas of the concrete can give pleasing designs (Figure 11.17).

Surfaces can also be ground and polished. *Terrazzo* is a decorative finish that consists of a facing made of cement (usually white) and marble chips. When the terrazzo has hardened, the surface is coarsely ground to expose the aggregate. Any air voids are filled in with cement paste, and the surface is finely ground and polished. Early power grinding, two to seven days after casting, is an alternative finishing technique for slabs that is used in Europe. The procedure uses low-speed grinders that produce a slip-resistant surface.

Various tools can be used to fracture the concrete at the surface. Scaling produces a fine texture at the surface. The scaler has three pneumatic chisels that rotate and fracture the concrete surface on impact. Bush hammering is often used to expose aggregate in the plane of the concrete surface, since mostly cement paste is removed. A bush hammer has a face, resembling a meat tenderizer, which strikes the surface repeatedly. Jack hammering or chiseling is done when the concrete is strong enough so that coarse aggregate is fractured as well as the paste. A variety of effects can be obtained by these methods. One common method is to cast ribs (or reeds) into the concrete surface using a form liner; the ribs are abraded, bush hammered, or chiseled to produce a striated effect (Figure 11.18).

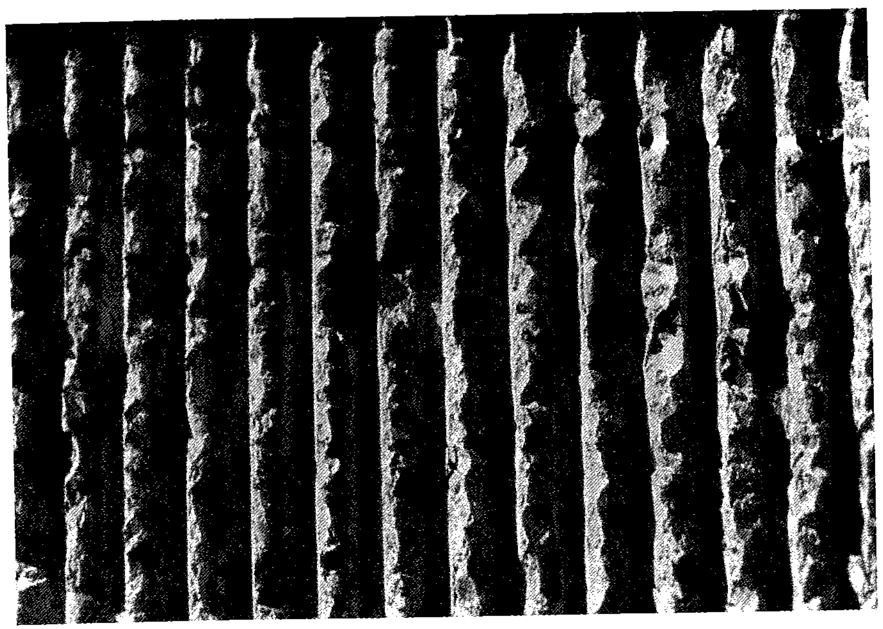


FIGURE 11.18

Texture produced by bush hammering vertically cast ribs. (Photograph courtesy of Portland Cement Association.)

11.5 SPECIAL ASPECTS OF HANDLING CONCRETE

Seasonal Concreting

When placing concrete under the extreme weather conditions that may occur in summer or winter, certain aspects of concreting can be adversely affected and special precautions are needed to ensure quality concrete. Here we discuss problems of concrete mixing and placing, while special curing procedures are discussed in Chapter 12.

Hot Weather Concreting The main problems associated with concreting in hot weather are high concrete temperatures and loss of moisture during placement and finishing. High concrete temperatures increase the water requirements to maintain a given slump; decrease setting times, and hence the time available for placement, consolidation, and finishing; increase the danger of plastic shrinkage; and lower the ultimate strength. The optimum concrete temperature should be in the range 10 to 15°C (50 to 60°F), and even lower for mass concrete where there is an increased risk of thermal cracking. This can seldom be achieved in summer without artificial cooling. Concrete temperatures should not exceed 30 to 33°C (85 to 90°F) and should be kept as low as is practical.

The effect of concrete temperature on the amount of water needed to maintain a given slump (known as "water demand") is illustrated in Figure 11.19. An increase of 5°C (9°F) will require an additional 3.3 kg/m^3 (5.6 lb/yd^3) of water to maintain a 75-mm (3-in.) slump. [This is about 6 lb/yd^3 for an increase of 10°F.] For a compressive strength of 35 MPa (5000 psi), an increase in concrete temperature from 20 to 30°C

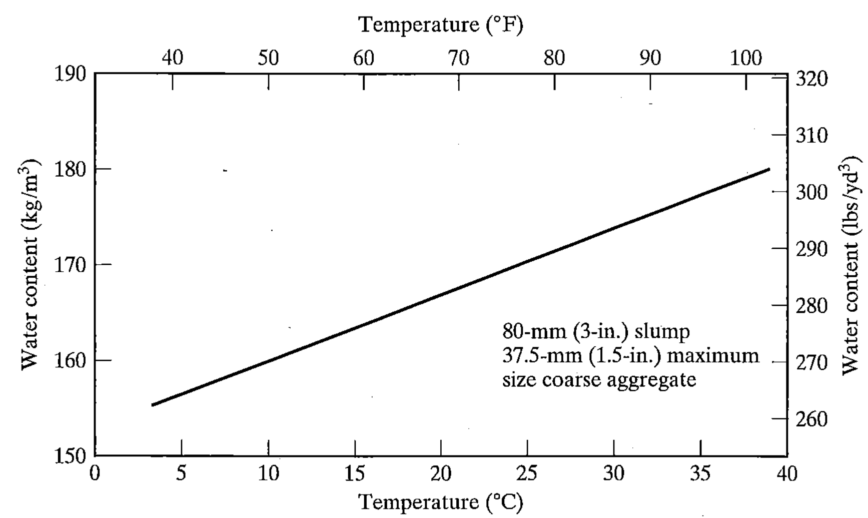


FIGURE 11.19

Water requirement for a concrete mix as affected by temperature. [Adapted from *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975).

(68 to 86°F) will translate into a loss in strength of about 2.5 MPa (360 psi) if the only adjustment to account for the temperature rise is an increase in water content to maintain the concrete slump. Unfortunately, it is not unusual for concrete mixes to be designed in the laboratory at 20°C (68°F) and used throughout the year with no adjustments other than changes in the water content. Retarders and water reducers can counteract some of the problems by reducing the rate of hydration (and thus the concrete temperature) or increasing the slump without increasing the *w/c* ratio. The time period during which these admixtures are effective is shortened as the concrete temperatures rises.

Concrete temperatures can be regulated by controlling the temperature of the ingredients. The contribution of each constituent is determined by its temperature, specific heat, and weight fraction. This is the basis of the relationship

$$T_{\text{concrete}} = \frac{0.22(T_a W_a + T_c W_c) + T_a W_{wa} + T_w W_w}{0.22(W_a + W_c) + W_{wa} + W_w} \tag{11.1}$$

which can be used to calculate the concrete temperature, in either °C or °F. In Eq. (11.1), W_a, W_c, W_{wa}, and W_w are the weights (kg or lb) of aggregate, cement, aggregate moisture, and mixing water, and T_a, T_c, and T_w are the temperatures of aggregate (including its moisture), cement, and mixing water, respectively. Using this equation, it can be seen that cooling the water is the most effective means of lowering the concrete temperature and is easier than cooling the aggregate. The high specific heat of water offsets its small weight fraction so that its contribution is about the same as that of the aggregate. Using ice is even more effective, since heat is absorbed in melting the ice. Equation (11.1) then becomes

$$T_{\text{concrete}} = \frac{0.22(T_a W_a + T_c W_c) + T_a W_{wa} + T_w W_w - F_i W_i}{0.22(W_a + W_c) + W_{wa} + W_w + W_i} \tag{11.2}$$

where W_i is the weight of ice and F_i is a function of the latent heat of fusion, the specific heat, and the freezing temperature of water (76.6°C for SI units and 112°F for customary units). Ice must be completely melted before mixing is complete. Other simple precautions can be used to keep the temperatures of all materials as low as possible, such as storing materials in the shade or in cooled bins; spraying aggregates with water or covering with white reflective sheeting; and painting water tanks and lines white. The temperature of the cement is seldom a bother, since both its specific heat and weight fractions are low. A change in cement temperature of 9°C is needed to change the concrete temperature 1°C, and only if cement is delivered from the plant still warm are problems likely to occur. Normally, cement has cooled before being shipped, but sometimes in the summer, when the demand for cement is at its greatest, it may be shipped almost as soon as it is made.

Other precautions in the field can help keep the concrete cool. Concrete should be mixed for the minimum required time and kept shaded from direct sun as much as possible while being transported and placed. It is better to confine actual concreting to early or late in the day, and it may be desirable to consolidate the concrete in shallower layers than would be used at lower temperatures to reduce the chance of developing cold joints. Forms, reinforcing steel, and subgrade should be wetted down thoroughly just before placement. This not only helps keep the area cool, but it minimizes loss of moisture from the concrete by absorption or evaporation, which can cause plastic shrinkage cracking. The concrete should also be protected from drying winds, which increase the rate of evaporation, further contributing to plastic shrinkage cracking. Figure 11.20 provides a method to estimate the rate of evaporation based on air temperature, relative humidity, concrete temperature, and wind velocity. Rates of evaporation above 1.0 kg/m²/h (0.2 lb/ft²/h) require protective measures. Some concretes, especially those containing pozzolans or entrained air, will undergo plastic shrinkage cracking at lower rates of evaporation due to reduced bleeding rates, requiring protective measures starting at evaporation rates of 0.5 kg/m²/h (0.1 lb/ft²/h). Plastic shrinkage can be controlled by reducing wind velocities with windbreaks, by reducing the temperature of the concrete, or by increasing the rate of setting of the concrete (if construction practices allow it). By far, the most effective method of control is to ensure that the concrete surface remains wet until the surface has been finished and routine curing begun. Temporary wet coverings, waterproof sheeting, or a fog spray to limit the rate of evaporation are appropriate methods. (Plastic shrinkage cracking is discussed in more detail in Section 16.1.)

In addition to plastic shrinkage cracking, rapid evaporation may result in crusting of the upper surface of slabs, which can make the slabs appear ready for finishing before the slab can adequately support the finishing operations. The concrete deforms without the formation of local indentations in the surface, resulting in a slab with a rolling (nonflat) surface. This problem can occur at all temperatures (including cold weather) when the rate of evaporation removes water from the surface faster than it is replaced by bleed water and before the slab has adequately set. The same measures used to limit plastic shrinkage cracking will help limit crusting.

Cold Weather Concreting In the winter months, the concern is the exact opposite: the need to maintain concrete at an adequate temperature and to prevent the concrete from freezing early in its life. Recommended minimum concrete temperatures are given in Table 12.1 (Section 12.1); these make allowances for the cooling effect of the environment. The same considerations apply to warming concrete as to cooling it, but in reverse. The use of hot water is the easiest way of raising concrete temperatures. Warming aggregates also helps, as does storing them in heated enclosures. The use of frozen aggregates is not recommended because of difficulties in batching and increased variations in moisture contents.

While the temperatures recommended in Table 12.1 are minimums, temperatures much above 21°C (70°F) are not recommended for the following reasons: (1) Higher temperatures will only result in the difficulties inherent in hot-weather concreting; and (2) if adequately protected, concrete cast at lower temperatures will have superior long-term properties because of inherently lower water requirements and improved hydration. Figure 12.2c shows that concrete initially mixed at 4°C (39°F) and cured at 21°C (70°F) will have higher long-term strength than concrete mixed at higher temperatures. Increased temperatures would only sacrifice these improved properties.

Lower-temperature concrete will not only require less water for a given slump, but will maintain that slump longer, bleed for a longer period, and take longer to set (a major problem when finishing flatwork). To account for these differences from higher-temperature

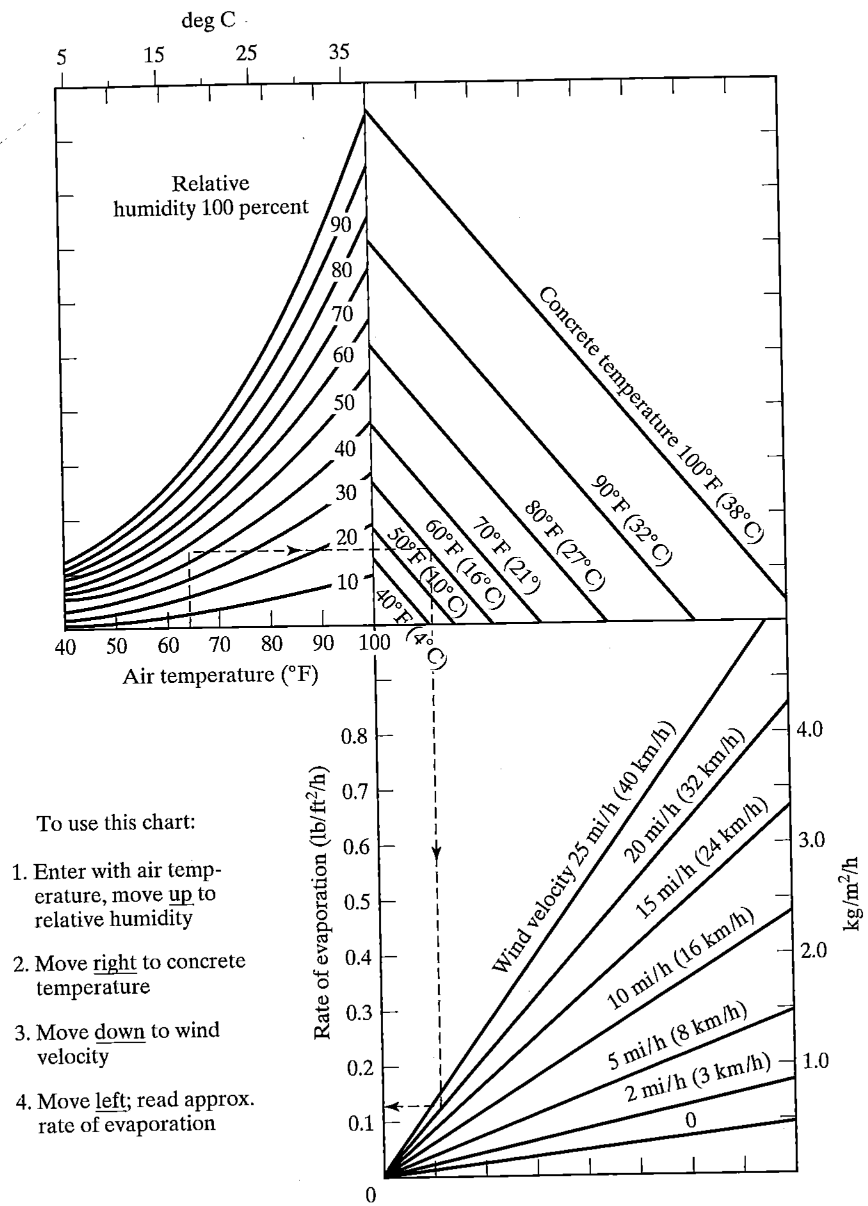


FIGURE 11.20

Chart to estimate the rate of evaporation of water from freshly placed concrete. [From *Design and Control of Concrete Mixtures*, 13th ed., Portland Cement Association, Skokie, IL (1994).]

concrete, the concrete should be mixed with a lower slump, and care should be taken not to overwork the concrete in the presence of bleed water. If excess bleed water remains on the surface of a slab, it can be removed by dragging a hose across the concrete.

As with hot-weather concreting, if bleeding is limited by high air contents or the use of pozzolans, or if drying conditions prevail, the upper surface of a slab can crust or appear ready for finishing before the slab has set up enough to support the finishing operation. Early finishing will cause sealing of the surface, which prevents subsequent bleed water from escaping and results in a weakened, high *w/c* ratio paste just below the surface, while forcing coarse aggregate away from the wearing surface.

Concrete should never be placed on frozen subgrade. Heat loss from the concrete may cause the lower part of a slab to freeze, while uneven settlement or subsequent thawing may cause cracking.

Proper curing practices (described in Chapter 12) should be carefully followed. Since low-temperature concrete gains strength more slowly at early ages, the strength of the concrete in the structure should be monitored to determine when it is safe to remove forms or shoring (temporary members used to support concrete once forms have been removed) and when it is safe to support continuing construction loads on the new concrete. Major failures have occurred when the effect of low temperature on the rate of concrete strength gain has been ignored. The maturity concept, discussed in Chapter 14, provides an excellent tool to account for the rate of strength gain. The concrete should be protected from freezing for the first day or two. If fresh concrete is frozen, it is permanently damaged, and subsequent normal curing will not restore its properties. Hardened concrete should be allowed to partially dry for maximum protection before being exposed to freezing. Air-entrained concrete should be used if the hardened concrete will be exposed to freezing on the job site, even if it will not be exposed to freezing once the structure has been placed in service.

Formwork

Formwork is a very important aspect of concrete construction. A detailed discussion is not presented here simply because there is not space to deal adequately with the subject. Whole books have been devoted to formwork, and those recommended in the bibliography at the end of the chapter should be consulted. Nevertheless, the design and construction of formwork can be crucial to the successful implementation of a design and the economics of construction. Formwork is the sole support of fresh concrete and must be designed to withstand some structural loads during construction, including the effects of materials and equipment as well as the lateral pressures induced by fresh concretes. Structural failure of concrete formwork is the largest single cause of construction failures, and a number of states now require formwork to be designed by a registered engineer. Formwork should be watertight, rigid, and properly aligned.

The choice of formwork material is governed solely by questions of cost and desired surface finishes. The use of prefabricated reusable forms is an increasingly cost-effective strategy, while on the other hand, permanent forms that become the exterior surface can be used to advantage.

Inspection of Concrete

Frequent and thorough inspection of all concrete operations is also an important part of the whole operation. The good inspector has a good background of concrete and concrete operations, is conversant with the job specifications, has plenty of common sense, and gets along well with people. Adversary relationships between contractor and engineer will not produce quality concrete; problems must be solved with mutual cooperation. The inspector should exercise good judgment when interpreting test results and checking compliances with specifications.

Inspection should cover all aspects of concrete construction and starts with an inspection of the batching plant and its operation. Upon delivery of concrete to the job site, the concrete should be tested to ensure that it is of uniform quality and conforms to specifications. Standard ASTM test methods should be used unless special tests are specified. Before the concrete is placed, the inspector will ensure that formwork is constructed according to the plans, is properly aligned, is sturdy and watertight to resist concrete pressures, and is built of clean, sound materials. The inspector has also checked that the reinforcement is laid in correct location and spacing and with the required splicing of overlapping bars, and is free of rust. During placement, the various operations should be regularly monitored to ensure that the concrete is not tending to segregate, is properly consolidated, and special precautions appropriate to the conditions are followed. Finally, the inspector should ensure that the concrete is adequately cured.

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PROBLEMS

11.1 Why is it important that concrete be efficiently mixed?

11.2 Compare the advantages and disadvantages of central-mixed and truck-mixed concrete.

11.3 What precautions should be taken when transporting concrete if the job site is a long way from the batching plant?

11.4 Under what circumstances should it be permissible to add material to the concrete mix on the job site?

11.5 What precautions should be taken when belt conveyors are used to transport concrete at a construction site?

11.6 What are the special requirements of mix design when concrete is to be pumped?

11.7 How would you guard against concrete segregation on the job site?

11.8 What are the advantages and disadvantages of slipforming?

11.9 Describe a method for placing concrete under water, citing any special requirements and precautions.

11.10 What are the effects of (a) undervibration; (b) overvibration?

11.11 How should the method of compaction change as the workability of concrete decreases?

11.12 What is the difference between a cold joint and a construction joint?

11.13 Describe the purposes of the procedures used in finishing concrete flatwork.

11.14 What is honeycombing, and what causes it?

11.15 What is the purpose of controlling the temperature of the concrete ingredients (a) in summer; (b) in winter?

11.16 What is the function of an inspector on the job site?

CHAPTER 12

Curing

Concrete must be properly cured if its optimum properties are to be developed. An adequate supply of moisture is necessary to ensure that hydration is sufficient to reduce the porosity to a level such that the desired strength and durability can be attained and to minimize volume changes in the concrete due to shrinkage. Concrete structures rarely fail because the specified design strength is not attained, but inadequate strength at the time the forms are stripped may cause problems. Premature cessation of moist curing and removal of formwork during winter construction have more than once been the cause of costly construction failures due to the loading of under-strength concrete. Concrete needs time to gain strength, even when good curing methods are being used, and the strength should be checked prior to form removal. The loss of potential durability in the long term due to inadequate curing is a more widespread and insidious problem, since the maintenance-free service life is reduced. It is thus essential to apply proper curing to the "skin" of the concrete ("covercrete"), since it is this covercrete that will be directly exposed to the environment and possible chemical or physical attack. In recognition of this point, some authorities pay for curing as a separate item on construction projects, rather than just specifying that it be done.

12.1 CURING AT AMBIENT TEMPERATURES

Parameters Affecting Curing

As mentioned earlier, water must be supplied for the hydration of cement. Although cement paste will in practice never completely hydrate, because the largest grains become covered with a thick layer of C–S–H that inhibits reaction, the aim of curing is to ensure as much hydration as possible at reasonable cost. Also, in pastes with lower *w/c* ratios, such as those in high-strength concretes, self-desiccation can occur during hydration and thus prevent further hydration, even when water is supplied externally (see Chapter 4). Theoretically, there is enough water in concrete to ensure complete hydration without additional water being supplied if the *w/c* ratio is 0.42 or greater. However, in practice, water is lost from the paste by evaporation, or by absorption of water by aggregates, formwork, or subgrade. To reduce this absorption, the formwork and subgrade should be dampened prior to placement. (The question of absorption by the

aggregates is discussed in Chapter 10.) Once enough moisture is lost from the concrete, so that the internal relative humidity drops below about 80% either by evaporation or self-desiccation, hydration will stop and strength development will be arrested. The strength of the concrete will thus be reduced below its potential, and this reduction will be greater in the case of high-strength concrete (low *w/c* ratio) than it is in the case of low-strength concrete (high *w/c* ratio). Therefore, it is desirable to provide additional moisture during curing to ensure maximum hydration.

Figure 12.1 shows the effect of limited moist curing on the development of the compressive strength of concrete. On the cessation of moist curing, the rate of strength gain slows down as water is lost from the concrete, and further strength gain soon ceases. A 3-day period of moist curing will only allow the concrete to reach 75 to 80% of the potential 28-day strength which can be achieved with continuous moist curing. Furthermore, none of the additional 25 to 30% of strength gain that can accrue beyond 28 days will be realized.

Interrupted Curing Exterior concrete will be exposed to wetting and drying repeatedly during its lifetime, so that the opportunity for continued hydration still exists. Although it is true that resaturated concrete will resume its interrupted hydration, the amount of strength developed is not as high as it would have been if moist curing had not been interrupted. Moreover, it is not easy to fully resaturate concrete except by means of prolonged immersion, so that the amount of additional hydration will be variable and unpredictable and may be quite small. Interrupted moist curing of young concrete is particularly undesirable. In addition to the reasons just discussed, intermittent moist curing will also subject the concrete to wetting and drying cycles at a time when

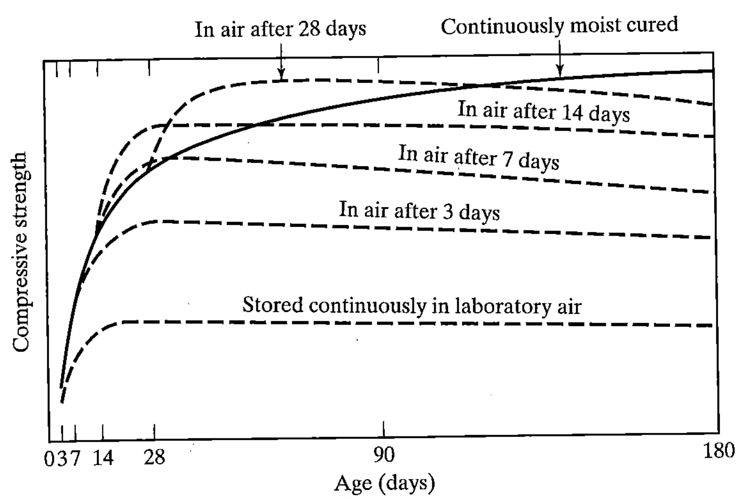


FIGURE 12.1

Schematic representation of the compressive strength of concrete dried in laboratory air after preliminary moist curing. [Adapted from *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975); W. H. Price, *Journal of the American Concrete Institute*, Vol. 47, No. 6, pp. 417–432 (1951).]

the concrete is weak enough to be susceptible to tensile stresses that may develop during drying. In the summer months, interrupted moist curing may subject the concrete to thermal length changes, which could also cause cracking.

Effect of Relative Humidity Concrete will hydrate even if it is not in a fully saturated condition. This is because water is held in larger capillary pores by surface tension forces below 100% RH. The cement can draw on these water reservoirs for further hydration, but the rate will become slower as the relative humidity maintained within the paste is lowered. The partial emptying of the capillary pores interferes with the flow of water in the pore system. Water is used first for hydration of cement within its localized area of the paste, and those areas that hydrate more rapidly (fine cement particles present, for example) will become starved of water. Additional water will move to those areas rapidly in a fully saturated system, but much more slowly in a partially saturated system. In the latter case, this will become the rate-determining step. Therefore, concrete that is sealed against moisture loss hydrates and gains strength more slowly than concrete that is continuously moist cured under water. Water cannot enter the sealed concrete, and the consumption of water during hydration lowers the internal relative humidity, slowing the rate of hydration. If the humidity falls below 80% RH, hydration will cease altogether, and this can happen in sealed low *w/c* ratio concretes. At *w/c* ratios < 0.3, this can occur even in concretes that are kept under water, due to their very low permeabilities. This may also occur in concretes exposed to an environment with a lower external relative humidity.

The level of the external relative humidity has the most influence on strength development through its effect on drying of concrete in its early life. The effect of drying is most severe in fresh concrete, and those factors are discussed in Chapter 16. The rate of drying of the concrete will have some influence on the residual strength gain after moist curing has been stopped. The surface temperature, wind velocity, and relative humidity are all important factors, as is the thickness of the concrete.

Effect of Temperature Figure 12.1 refers to curing at 21°C (70°F), but temperature affects the rate of strength development, and therefore the length of curing that is necessary. Figure 12.2a shows the effect of curing temperature on strength development after 1 day and 28 days. The increased early strength at higher temperatures is due to the fact that the cement simply hydrates more rapidly, but the decrease in later strength is not so easily explained. It is believed that neither the chemical nor the physical structure of the hydration products is radically changed by the hydration temperature up to about 45°C (115°F). The detrimental effect appears to be due to the fact that it produces a nonuniform distribution of hydration products, leaving weak zones in the cement that govern the strength. Figure 12.2b shows the effect of curing temperature on strength development to times of 90 days and indicates the relative length of curing times required to reach a specified strength at different temperatures. The effect of placing concrete at different temperatures and then holding it at 21°C during the curing period is shown in Figure 12.2c, where the same detrimental effect of higher temperatures is readily seen. Hydration occurs down to about −10°C (14°F). We should note that curing at low temperatures can result in a higher ultimate strength, even though the initial rate of strength development is low. Thus, as a rule, the higher

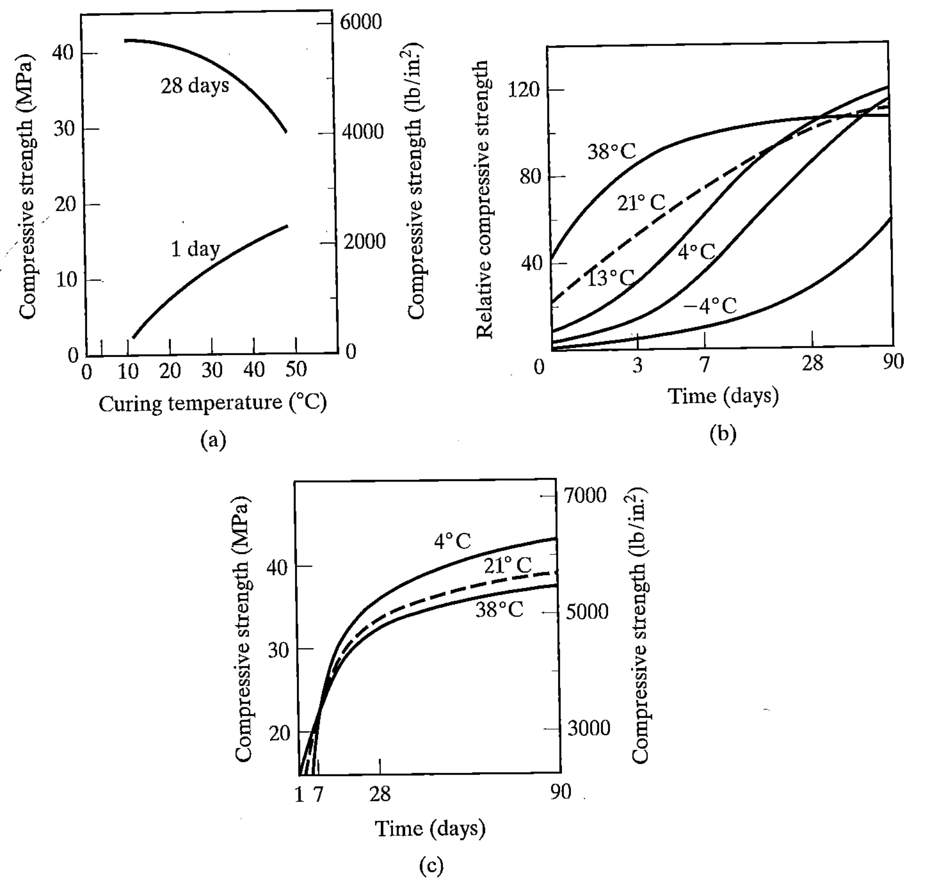


FIGURE 12.2

Effect of temperature on compressive strength development: (a) Comparison of 1-day and 28-day strengths (constant curing temperature). [Adapted from G. J. Verbeck and R. A. Helmuth, *Proceedings, Fifth International Symposium on the Chemistry of Cement, Tokyo*, Vol. 3, pp. 1–32 (1968).] (b) Curing temperature maintained continuously. (c) Initial concrete temperature, curing temperature maintained at 21°C. [Adapted from W. H. Price, *Journal of the American Concrete Institute*, Vol. 47, No. 6, pp. 417–432 (1951).]

the initial temperature of the concrete, the lower its later strength. For 28-day strengths, the optimum initial temperature is about 4°C (39°F) for a Type I cement, as seen in Figure 12.2c. In general, the tensile strength is similarly affected by the curing parameters.

Time of Moist Curing

The most desirable objective is to continuously moist cure the concrete as long as possible—ideally, even after the concrete has attained its specified strength. Obviously, this is seldom a practical proposition, and some compromise must be accepted to allow acceptable construction schedules and to minimize costs. It can be seen from Figure 12.1

that moist curing for about seven days will ensure that the 28-day moist-cured strength will eventually be reached. The ACI Standard Practice for Curing Concrete (ACI 308) suggests seven days of moist curing for most structural concrete or the time necessary to attain 70% of the specified compressive or flexural strength, whichever is less. For unreinforced mass concrete, minimum curing times should be longer: two weeks, or three weeks if a pozzolan is used. These longer times allow for the slower strength development of a low-heat cement and of the cement–pozzolan reaction.

These recommendations are for concretes placed and cured at temperatures above 4°C (39°F). At lower temperatures, the slow rate of hydration will mean that strengths may well be too low even after seven days of curing. The combined effects of time and temperature on the rate of hydration, and hence strength development, can be estimated using the maturity concept (see Chapter 14).

The danger of freezing is always present at low temperatures. Concrete should not be allowed to freeze until it has developed some strength (about 3.5 MPa or 500 lb/in.²). Generally, 24 h of moist curing at 4°C (39°F) will not provide this protection without the use of insulation during curing to conserve the initial heat of the concrete and its heat of hydration and thereby keep it out of danger. Additional heat will also be required in more severe cold.

Effect of Carbon Dioxide

In the manufacture of some precast components (e.g., concrete block), the units are exposed to high levels of carbon dioxide, and this results in a more dimensionally stable product, as well as an increase in strength. This kind of curing is only effective with relatively porous concrete with a thin cross section to allow reasonable penetration of carbon dioxide throughout the concrete (see Section 16.2).

Methods of Curing

There are many methods and materials that can be used for moist curing of concrete. These can be divided into two groups:

1. Water curing—those that supply additional moisture as well as prevent moisture loss.
2. Sealed curing—those that prevent loss of moisture only.

Water Curing Supply of water to the concrete can be accomplished by ponding, spraying, or sprinkling, or by use of saturated coverings. However, whichever system is used, the beginning of water curing should start before the end of the induction period (Chapter 4); that is, it should precede the time of initial set (ASTM C 403) by about 1 hour. This practice is necessary to maintain the water within the concrete (the interstitial water) as a continuous liquid system, making it easier for the external water to enter into the large capillaries of the concrete that begin to empty due to self-dessication. If water curing is delayed, menisci will develop in the capillary pores, and the water will then no longer be able to penetrate into the concrete as easily or completely. This situation could lead to local areas of self-dessication and autogenous shrinkage (Section 16.2). This issue becomes increasingly important as the initial w/c ratio is

decreased. (Of course, even late application of water curing is better than no curing at all.) For concretes containing fly ash or slag, which gain strength more slowly, water curing should continue for a longer period, to ensure that there is enough water present for the pozzolanic reactions to take place.

Ponding is a thorough method of curing horizontal surfaces and involves maintaining a layer of water on the surface by means of earth or sand dikes. This method is relatively labor intensive and requires considerable supervision. Keeping concrete moist by means of a fine spray is another excellent method when water is plentiful and runoff is no problem. It can be used on both vertical and horizontal surfaces. A fine spray should be used, such as is provided by most lawn sprinklers or a soil-soaker hose. Ideally, spraying should be continuous, but if it is intermittent, care must be taken to see that the concrete does not dry out between applications. Spraying should not be too vigorous or erosion of the concrete surface may occur. Thus, curing by spraying also requires careful supervision.

The use of coverings that can hold quantities of water in addition to preventing evaporation is another means of water curing. Burlap and other absorbent materials are widely used and can be applied to both horizontal and vertical surfaces. Damp earth, sand, straw, or sawdust have been used on horizontal surfaces, but these materials have been largely supplanted by labor-saving curing procedures (see the next section). These materials can double as insulation during the winter by topping off with a dry layer. Coverings of this type require periodic moistening, as they will tend to dry out, so regular supervision is still required. Staining of concrete surfaces sometimes occurs by contact with soluble organic chemicals that may leach from the material, and these may occasionally retard setting or hardening of the surface. The leaching of tannic acid from sawdust with a high tannin content is an example.

Sealed Curing Waterproof paper, plastic sheeting, and curing membranes are the most widely used materials for sealed curing. Their convenience and lower labor requirements have led them to displace the more traditional water-curing methods in many instances. Formwork can also act as an evaporation barrier, but wooden formwork will absorb moisture from the concrete if not kept damp during the curing period.

Waterproof paper or plastic sheeting should be applied as soon as the surface has hardened sufficiently to prevent surface damage and after the concrete has been thoroughly wetted. Plastic sheeting is more versatile in that it is more flexible and can be used to cover more complex shapes. Plastic films can be bonded to absorbent materials, which help to retain and redistribute moisture that evaporates from the concrete and condenses on the cover. In this way, the moisture can be returned to the concrete and improve its curing. Both paper and plastic can be made white to reflect sunlight and reduce absorption of heat in summer or can be colored black to increase absorption of heat in winter. Specifications for sheet materials are given in ASTM C 171.

Liquid membrane-forming curing compounds have become very popular in the curing of concrete pavements and floors and can also be used on vertical surfaces. From the concrete's point of view, these compounds provide the least effective method of curing, since they do not entirely prevent evaporation from the concrete, and it is

best if they are applied after some initial period of moist curing. In modern pavement construction, however, the application of a curing compound immediately follows the slipform paver.

Membrane-forming compounds are formulated from aliphatic alcohols, resins, waxes, or synthetic rubbers dissolved in a volatile solvent or emulsified in water. Upon removal of the solvent by evaporation, an almost impermeable membrane forms on the surface and seals the concrete against moisture loss. Pigments can be added to the formulation: a white pigment in hot weather to reduce absorption of heat, and gray or black for cold weather. The use of a pigment is advisable so that it can be readily seen whether a complete covering has been applied. Curing compounds should not be used when a topping or overlay will subsequently be laid, or if the surface is to be painted, since the membrane will interfere with the bonding of these materials to the surface. Nor should curing compounds be used in the fall during the construction of pavements that will be exposed to deicing salts, since the membranes retard the air drying that is needed to improve the salt-scaling resistance of the surface. Specifications for membrane curing compounds are given in ASTM C 309.

Comparison of Curing Methods As mentioned earlier, water curing is better than sealed curing since it protects the concrete against self-desiccation. This is particularly important when using low *w/c* ratio concretes ($w/c < 0.40$), where self-desiccation can occur rapidly. The efficiency of sealed curing depends very much on the thickness of the covering and the integrity of the covering; care must be taken to ensure that separate sheets are properly overlapped and the joints sealed against moisture loss. Materials for sealed curing can be evaluated by ASTM C 156.

The selection of a curing method should be carefully considered where self-desiccation can readily occur. This is strikingly illustrated in the case of curing expansive cements, where the formation of ettringite creates a high demand for water during hydration. As can be seen in Figure 3.11 (Section 3.3), the choice of an inadequate method will reduce the potential expansion and hence the protection against shrinkage cracking. This illustrates that curing compounds form a somewhat less effective vapor barrier than plastic sheeting, although a good membrane can provide an effective seal.

Curing in Special Situations

Mass Concrete In the curing of mass concrete, temperature control becomes as important as moisture control. It is important to achieve a constant and uniform temperature throughout the mass as soon as possible after placement. According to ACI 308, Standard Practice for Curing Concrete, the internal temperature should not rise more than 11 to 14°C (20–25°F) above the mean annual ambient temperature. An internal cooling system may be needed to ensure this. Since low-heat cements are generally used, the period of moist curing must be extended.

Hot-Weather Concreting Hot, dry conditions that may prevail are very damaging to freshly placed concrete, and particular attention must be paid to curing procedures under such conditions. The avoidance of excessive plastic shrinkage is imperative. Concrete should be protected from drying conditions, direct sun or wind, and should be kept covered temporarily after finishing and before moist curing is commenced.

Curing materials should be used that will reflect sunlight so that concrete temperatures will not become too high. Water curing is most desirable, and care should be taken to keep the concrete moist at all times because of possible stresses imposed on the concrete by alternate wetting and drying or by contact of cold water on a warm concrete surface. ACI Report 305R addresses the special problems of hot-weather concreting.

Cold-Weather Concreting During cold weather, special precautions should be taken in curing, as outlined in the ACI Report 306R. The problems associated with temperatures below 4°C (39°F) are:

1. Freezing of concrete while saturated and of low strength.
2. Slow development of strength.
3. Thermal stresses on cooling to ambient temperatures (if the fresh concrete is heated during curing).

To help offset the problems of 1 and 2, the concrete temperature should not be too low when placed (see Table 12.1). If the ambient temperature is not too low, the heat of hydration, together with adequate insulation of formwork and exposed surfaces, should protect the concrete from freezing in its early life. Lower concrete temperatures can be specified for mass concrete because less heat is dissipated during the curing period. Concrete temperatures above 21°C (70°F) do not provide more protection from freezing because the loss of heat is greater during transporting and placing. Also, higher temperatures will require more mixing water and shorten the working time. Two days' protection is recommended when Type I cement is used and one day when Type III cement, or an accelerating admixture, is used. Batt insulation or special insulation blankets can be used; dry straw is also effective.

When temperatures are too low, insulation may not be sufficient to maintain a temperature that will give adequate strength development and prevent freezing. In this case, the concrete needs to be heated by an external heat source. This can be done by heating the formwork directly or by building an enclosure around the concrete and its formwork and heating the enclosure. Live steam is the best form of heating since moisture is also supplied, but "dry heat" is more commonly supplied through the use of combustion heaters, which should be vented outside. When these are used, special care must be taken to ensure that the concrete does not dry out. When concrete is externally heated during the curing period, it should not then be exposed directly to a low ambient temperature because of the danger of thermal shock. The concrete should be allowed to cool gradually; the maximum temperature drop during the first 24 h should not exceed that given in Table 12.1.

TABLE 12.1 Recommended Concrete Temperatures in °C (°F) for Cold-Weather Construction[a]

	Size of Section		
Condition of Placement and Curing	*Thin*	*Moderate*	*Mass*
Minimum temperature of fresh concrete, *as mixed* for air temperatures that are			
Above 0°C (32°F) ambient	16 (60)	13 (55)	10 (50)
−18 to 0°C (0° to 32°F) ambient	18 (65)	16 (60)	13 (55)
Below −18°C (0°F) ambient	21 (70)	18 (65)	16 (60)
Minimum temperature, *as placed*	13 (55)	10 (50)	7 (45)
Maximum allowable drop in concrete temperature during first 24 h after end of protection	28 (50)	22 (40)	17 (30)

[a] Adapted from ACI Committee 306, "Cold Weather Concreting (ACI 306R-88)," *ACI Manual of Concrete Practice*, Part 2, American Concrete Institute, Farmington Hills, MI (2000).

12.2 CURING AT ELEVATED TEMPERATURES

Low-Pressure Steam Curing

Curing in live steam at atmospheric pressure dramatically increases the rate of strength development of concrete. Steam curing is used primarily for precast concrete products such as masonary block, pipe, prestressed beams, and wall panels, but can also be used for enclosed cast-in-place structures. In the precast concrete industry, steam curing allows increased production by a more rapid turnover of molds and formwork, shorter curing periods before shipment or prestressing, and less damage to the product during handling.

Maximum curing temperatures may be anywhere in the range 40 to 100°C (104 to 212°F), although the optimum temperature is in the range 65 to 80°C (150 to 175°F). The temperature will be a compromise between rate of strength gain and ultimate strength (see Figure 12.3), since, as we have already seen, the higher the

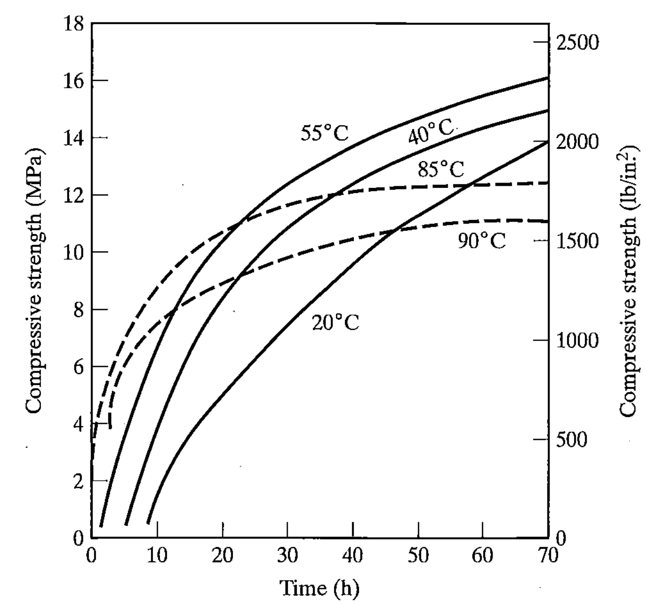


FIGURE 12.3

Effect of low-pressure steam curing on early strength development. These data were obtained on concrete steam cured immediately after casting, but it is preferable to delay steam curing a few hours; see text. [Adapted from *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975).]

initial temperature, the lower the ultimate strength. The optimum temperature depends on the application; the use of a lower temperature requires a longer curing period, but gives a better ultimate strength.

A word of caution must be added here. It has been found that concretes that have been subjected to high-temperature curing (up to 100°C) *may* suffer deterioration during later exposure to moisture at ordinary ambient temperatures. This phenomenon, known as *delayed ettringite formation* (see Section 18.3), is characterized by the development of "gaps" around some aggregate particles; these gaps often contain densely compacted ettringite (Chapter 4). This problem occurs primarily when cements are used that have a ratio of SO_3 to $Al_2O_3 > 0.5$.

The Curing Cycle The length of the total curing period must allow for controlled heating and cooling of the concrete. A typical curing sequence is given schematically in Figure 12.4. The concrete should be allowed to remain at room temperature for some time after molding before being exposed to steam, to allow the product to undergo some initial hydration and improve its stability; this will improve later strength. The optimum length of this presteaming ("holding" or "delay") period is generally 2 to 6 h; it depends on the type of cement, maximum curing temperature, and so on and should be determined for each application.

The rate of temperature rise and duration of the presteaming period are apparently interrelated. If a "presteaming" period is used, quite high rates of heating can be used (up to 33°C/h or 60°F/h), but in the absence of presteaming, the rate should not exceed 11°C/h (20°F/h). Generally, it is desirable to use as low a rate of heating as possible consistent with a convenient and economical curing schedule.

The time the concrete spends at the maximum temperature (the "soaking" time) determines the amount of strength gain during the curing period. The strength gain can be related to the time–temperature product (Figure 12.5) or to the "maturity" (see Chapter 14); thus, the higher the maximum curing temperature, the shorter the curing

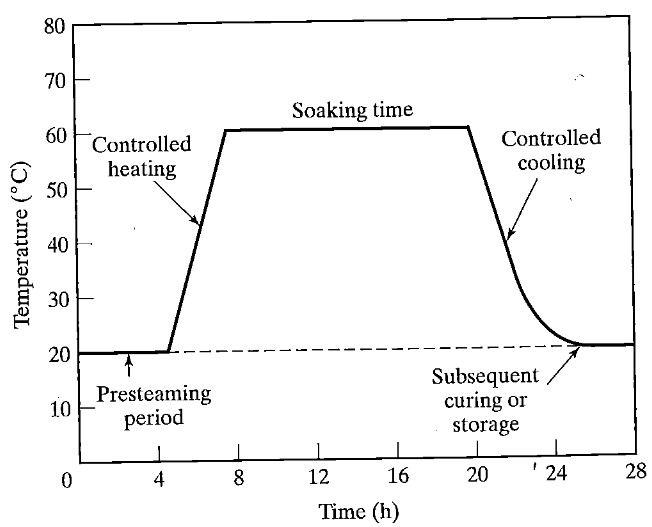
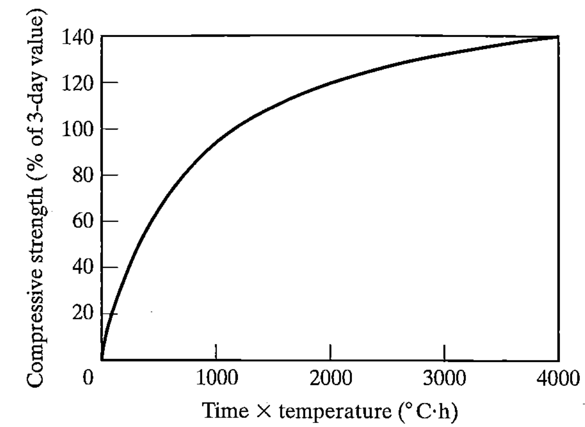


FIGURE 12.4

Typical sequence for low-pressure steam curing.

FIGURE 12.5

Influence of time–temperature product on strength development of low-pressure steam-cured products. (From R. W. Nurse, *Building Research Congress*, 1951, Division 2, Part D, p. 89, Building Research Establishment, Garston, Watford, U.K. British Crown Copyright, HMSO.)



time. Generally, as we have already observed, this higher rate of hydration will lead to a lower ultimate strength. Figure 12.3 shows that above 80°C (175°F), the effect of the high curing temperature on strength is noticeable before 3 days; but it has been suggested that by optimizing the "presteaming period" and avoiding high heating rates, the strength after 3 days can exceed the 28-day strength of normally cured concrete.

At the end of the curing period, the hardened concrete is much less susceptible to thermal shock than the fresh concrete was, and control of cooling rates is thus not as critical as control of the heating rates. Nevertheless, it is advisable to hold the rate of cooling in the range 22 to 33°C/h (40 to 60°F/h). Sometimes the steaming cycle is designed to provide only sufficient strength for safe handling of the product, and a period of additional moist curing at room temperature *(secondary curing)* is needed. Units can be placed in a fog room or merely stored in a moist condition under a waterproof covering for the additional time required. Products that have gained the desired strengths are generally allowed to dry before shipment; either air drying or accelerated drying using hot air can be used. Masonry block is often dried in this way; lowering the moisture content improves its dimensional stability and raises the compressive strength somewhat.

Properties of Concrete The characteristics of low-pressure steam-cured concretes do not differ markedly from those of concretes cured under ambient conditions. The hydration of the cement compounds proceeds more rapidly (C_3S is completely hydrated in three days at 100°C), but the hydration reactions are basically the same. C–S–H remains an amorphous material, although its composition changes with curing temperature. At higher temperatures, there is an increasing tendency for the calcium sulfoaluminates to become unstable and much more sulfate and alumina are incorporated into the C–S–H. All ASTM types of portland cement can be used in steam curing. The reactions of pozzolans and slag additions are also accelerated by the higher curing temperatures. Air-entraining admixtures, water reducers, and set-controlling admixtures can be used with steam curing.

The decrease in ultimate strength is believed to be a result of a less uniform distribution of hydration products in the paste because of the rapid initial hydration. This

is reflected in changes in pore-size distribution, particularly of the large capillary pores greater than 0.01 μm in diameter. This also results in a somewhat higher permeability, although the change is not great and durability is little affected by steam curing. Low-pressure steam-cured concretes generally show lower creep and shrinkage strains (about one-third less).

High-Pressure Steam Curing

If curing temperatures in excess of 100°C are desired, then saturated steam pressures must be allowed to develop, and a sealed enclosure must be used. The pressure vessel is generally known as an *autoclave*, and the term *autoclaving* is synonymous with high-pressure steam curing. The range of curing temperature used in autoclaving is 160 to 210°C (320 to 410°F) at steam pressures of 6 to 20 atm. The chemistry of hydration changes under these conditions and results in a product that has substantially different properties from products cured below 100°C. The most important improvements are

1. Products ready for use within 24 h—the strength is generally equivalent to 28-day strength under ambient curing.
2. Substantially less creep and shrinkage.
3. Better sulfate resistance.
4. Elimination of efflorescence (see Chapter 20).
5. Lower moisture content after curing.

High-pressure steam curing can only be used for precast concrete products. It can be used to advantage in the manufacture of specialty products, such as lightweight cellular concrete and calcium silicate (sand–lime) bricks.

The Curing Cycle The curing cycle is similar to that used in low-pressure steam curing and consists of a "presteaming" period, a "soaking" period, and controlled rates of heating and cooling. The rate of pressure release at the end of the soaking period should also be controlled. The length of the presteaming period depends on mix design, curing temperature, and other factors. The rate of temperature and pressure rise should be such that maximum temperature is reached in about 3 h. Sometimes a period of low-pressure steam curing is used before autoclaving (the two-stage process), in which case pressure buildup can be more rapid. Again, the length of soaking depends on the maximum curing temperature and the desired strength: 8 h at 175°C (350°F) is a typical period. Pressure release at the end of the soaking period should be quite rapid and should be complete in 20 to 30 min. This allows flash evaporation of moisture from the product, which helps to cool it. As in low-pressure steam curing, the exact curing cycle will be determined by a compromise between satisfactory development of properties and economic considerations.

Chemistry of Autoclaving If products containing only portland cement as the binder are autoclaved, very low strengths are obtained. Satisfactory strengths are achieved only when reactive siliceous material is added to the product, as can be seen in Figure 12.6. This occurs because, under the conditions of high temperature and pressure, the chemistry

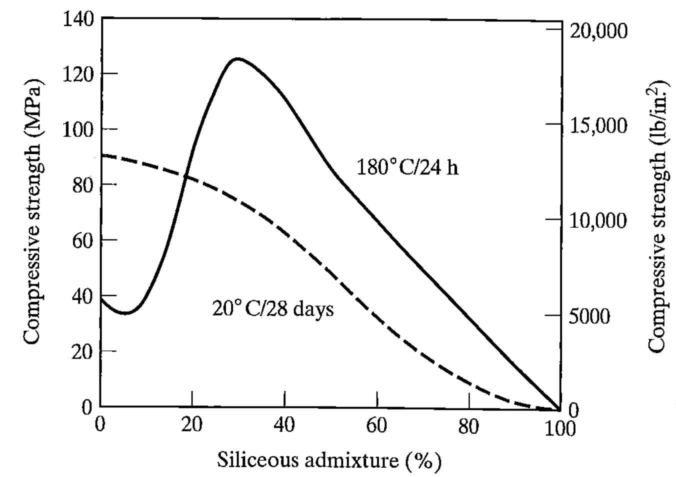


FIGURE 12.6
Effect of siliceous additions on the strength development of autoclaved concrete. [From C. A. Menzel, *Journal of the American Concrete Institute*, Vol. 31, No. 3, pp. 125–148 (1934).]

of hydration is substantially altered. C–S–H forms initially, as in normal hydration, but is rapidly converted to a crystalline product, α-dicalcium silicate hydrate, according to the reaction

$$\underset{\text{portland cement}}{(C_3S + C_2S)} \xrightarrow{H} \underset{\text{C–S–H}}{C_3S_2H_8} + \underset{\text{calcium hydroxide}}{CH} \rightarrow \alpha\text{-}C_2SH \qquad (12.1)$$

The conversion of C–S–H to α-C_2SH is accompanied by an increase in density and smaller volume of the solid phase, and therefore an increase in porosity. This change is analogous to the conversion reaction that causes eventual strength regression in concretes made with calcium aluminate cements.

The addition of reactive silica modifies the hydration sequence as shown in the reaction

$$\underset{\text{portland cement}}{(C_3S + C_2S)} + S \xrightarrow{H} \text{C–S–H} + CH + \underset{\text{silica}}{S} \rightarrow \text{C–S–H} \rightarrow \underset{\text{tobermorite}}{C_5S_6H_5} \qquad (12.2)$$

The presence of silica promotes the formation of poorly crystallized C–S–H, but one which has less lime than that formed in normal hydration. (Hence, it is not written as $C_3S_2H_8$.) This low-lime gel converts to another crystalline hydrate, called *tobermorite*, on continued heating. This change is not accompanied by a very large change in density, and hence the initial strength is maintained during the complete autoclave cycle. Equation (12.2) involves the pozzolanic reaction between reactive silica and lime, so pozzolanic materials such as fly ash or natural pozzolans can be used as the source of silica. However, at these elevated temperatures, less-reactive forms of silica will also act as pozzolans, and ground quartz (silica flour) can be used as the source of silica. The fine aggregate may also provide reactive silica, as in the case of expanded shale block; lower quantities of added silica are then needed. Hydrated lime may be used in place of portland cement, as in the case of sand–lime bricks.

At these high temperatures, calcium sulfoaluminate hydrates are not formed. Both the sulfate and aluminate apparently enter into the C–S–H and tobermorite structures. Alumina is said to increase the rate of crystallization of tobermorite. C_3AH_6 is occasionally observed, but is only a minor component.

Properties of Autoclaved Products It will now be appreciated that the optimum strength will occur when no α-C_2SH is formed, and this occurs at additions of 30 to 40% silica, depending on the exact composition of the cement used. Any silica in excess of that needed to form tobermorite does not react under these conditions, but becomes merely an inert filler that causes a decrease in strength proportional to its concentration. Hence, there is an optimum silica content, as seen in Figure 12.6. The pozzolanic reaction proceeds faster at higher temperatures, so that shorter curing times are needed. Prolonged autoclaving at 180°C or higher may cause the formation of other crystalline calcium silicate hydrates with a concomitant strength reduction. It is believed that even the complete conversion to tobermorite is not desirable and that there is an optimum ratio of amorphous to crystalline material for maximum strength.

The drying shrinkage of autoclaved concrete is only about one-third that of concrete cured under ambient conditions. This is due to the preponderance of crystalline tobermorite, which does not undergo the chemical and physical changes that occur in amorphous C–S–H as moisture is removed (see the discussion in Chapter 16). Since there is still a capillary pore network, surface tension forces operate on drying, and some shrinkage is observed; this is mostly reversible, however. The amount of shrinkage that occurs will depend on the proportion of amorphous C–S–H and the pore-size distribution; the presence of alumina will reduce shrinkage, and this is probably a result of its influence on tobermorite formation. Creep of concrete is similarly reduced.

Since no calcium sulfoaluminate hydrates are present in the matrix, resistance to sulfate attack is markedly improved. The absence of free lime must also play a role and is the reason that little efflorescence occurs in autoclaved products.

When properly cured, autoclaved products have a uniform light color and thus are very suitable for use with pigments. The color is more stable than in products cured by low-pressure steam or under ambient conditions. Defects caused by unsound materials or deleterious impurities also occur during autoclaving, rather than during service, and the defective components may be rejected before use.

There are few disadvantages to autoclaved products apart from the inherent technical limitations as to the kinds of products that can be so treated and the high capital cost of the plant. The bond strength between the concrete and the reinforcement is usually much lower (by about 50%), and the material tends to be more brittle than ordinary concrete.

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PROBLEMS

12.1 What are the parameters that affect curing?
12.2 Is interrupted curing detrimental to concrete quality?
12.3 Is it absolutely necessary to cure concrete at 100% RH?
12.4 What are optimum curing conditions for strength and durability? Are these the most economical conditions?
12.5 What are the advantages and disadvantages of curing compounds?
12.6 What special precautions must be taken when placing concrete in the summer?
12.7 What is the effect of freezing on very young concrete? How can freezing be avoided?
12.8 What are the advantages and disadvantages of (a) low-pressure steam curing; (b) high-pressure steam curing?
12.9 Why is it necessary to add silica to concrete products that will be autoclaved?
12.10 Why must both the rate of heating and the rate of cooling be controlled in the high-pressure steam-curing cycle?

CHAPTER 13

Response of Concrete to Stress

Concrete is a highly complex heterogeneous material whose response to stress depends not only on the response of the individual components but also upon the interaction between those components. The complexity of the interaction is illustrated in Figure 13.1, which shows typical compressive stress–strain curves for concrete and its mortar, cement paste, and coarse aggregate constituents. The coarse aggregate is a linear elastic brittle material, with a strength significantly above that of the concrete. The cement paste has a lower modulus of elasticity, but a higher compressive strength than does either the mortar or the concrete. The addition of fine aggregate to cement paste to obtain mortar results in an increase in the modulus of elasticity, but a reduction in strength. The addition of coarse aggregate to mortar, in the case illustrated, has little effect on the modulus of elasticity, but results in an additional reduction in compressive strength. Overall, the behavior of the concrete is similar to that of its mortar constituent, while the behavior of both the mortar and the concrete is significantly different from the behavior of either the cement paste or the aggregate.

The stress–strain curves in Figure 13.1 emphasize that concrete is a composite material, that is, a three-dimensional combination of two or more distinct materials with a definite interface separating the components. A multiphase material is expected to have different properties from the original components, and clearly concrete qualifies as such. At the macroscopic level, concrete consists of coarse aggregate embedded in a matrix of mortar; on a somewhat finer scale, the mortar consists of particles of sand embedded in a matrix of hydrated cement paste. On the microscopic scale, the hydrated cement paste consists of amorphous C–S–H and crystalline calcium hydroxide, containing an extensive network of capillary pores, that may be dry or filled with water, plus grains of unhydrated cement and grains of submicroscopic calcium sulfoaluminate hydrate crystals. On a still finer, submicroscopic scale, the C–S–H is a mixture of poorly crystallized particles of a variety of shapes and chemical compositions, with a more-or-less continuous system of gel pores, which also may be dry or partially or completely filled with water. The aggregates themselves are also

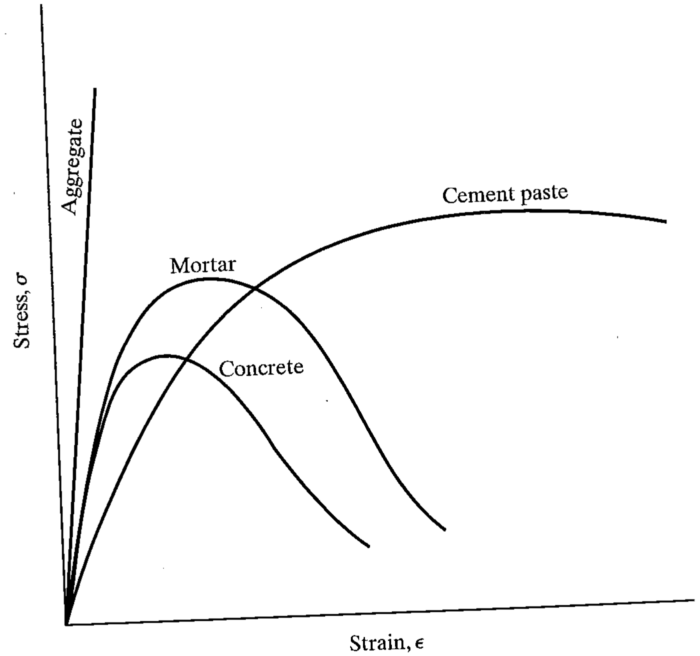


FIGURE 13.1

Typical stress–strain curves for aggregate, cement paste, mortar, and concrete.

generally composite materials, consisting of a mixture of different minerals with a well-defined porosity. Finally, the structure of the cement paste constituent at the boundary with aggregate particles differs from that of bulk cement paste, resulting in the formation of an *interfacial transition zone* (*ITZ*). The ITZ plays a role in the stiffness, strength, and permeability of cementitious materials containing aggregate due to its generally lower density and decreased strength compared to that provided by bulk cement paste. Often referred to as the "weak link" in concrete, the ITZ will be discussed at greater length in Section 13.1.

The balance of this chapter deals with the behavior of concrete under stress and includes discussion of both the theoretical and the practical aspects of that response. Emphasis is placed on material behavior under what is often referred to as "static" loading, that is, loading rates that are typically obtained under standardized test conditions. Even under such conditions, however, the response of concrete is sensitive to the nature of the tests, including the degree of restraint, the moisture content, and the rate of loading, since there is virtually no aspect of the mechanical behavior of concrete that is not affected by these factors.

13.1 INTERFACIAL TRANSITION ZONE (ITZ)

Traditionally, concrete has been modeled as a two-phase composite material, consisting of inert aggregate particles, with a wide size distribution, embedded in a *uniform* matrix of hydrated cement paste. For normal aggregates and concretes, it is then assumed that the mechanical and transport (diffusion, permeability) properties depend primarily on the *w/c* ratio. For normal engineering purposes, such a model provides a

sufficient degree of accuracy. However, it represents a considerable oversimplification of the true structure of concrete. It is well known that there is a thin zone surrounding the aggregate particles in which the structure of the cement paste is quite different from that of the "bulk" paste farther away from the physical interface, in terms of morphology, composition, and density. This zone, typically 20–40 μm thick, is referred to as the interfacial transition zone, or ITZ. As discussed in Chapter 4, the structure of the ITZ is different from that of the bulk paste in the following respects:

- There is less unhydrated cement.
- There is a higher porosity (and hence a lower density). The pores are generally larger than those found in the bulk paste.
- There is less C–S–H.
- There are large, oriented crystals of calcium hydroxide.
- There is generally a greater concentration of ettringite.

Two schematic models of the ITZ are shown in Figure 13.2.

Though the ITZ itself is quite narrow, it occupies a relatively large proportion of the cement paste. In a typical concrete, the mean spacing between aggregate particles is 75 to 100 μm. Assuming a 40-μm thickness for the ITZ, it has been estimated that the ITZ makes up 20–40% of the total volume of the cementitious matrix.

Effect of the ITZ on Mechanical Properties

Currently, there are no standard test methods by which the strength of the ITZ or, more importantly, the *bond strength* between the aggregate particles and the ITZ can

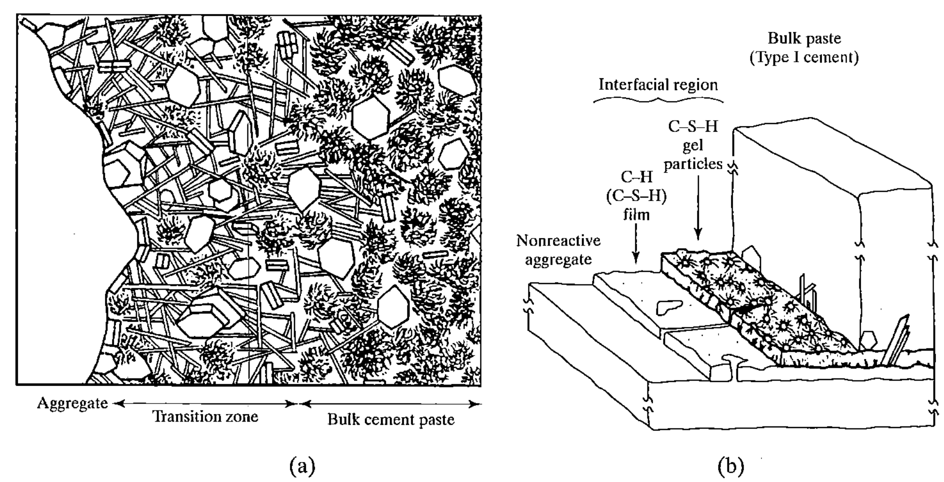


FIGURE 13.2

Two models of the ITZ: (a) diagrammatic representation [From P.K. Mehta and P.J.M. Monteiro, *Concrete, Structure, Properties, and Materials*, 2nd ed., Prentice-Hall, Englewood Cliffs, NJ (1993)]; (b) schematic of the ITZ [From C.A. Langton and D.M. Roy, *7th International Congress on the Chemistry of Cement,* Editions Septima, Paris (1980)].

be measured. The bond strength depends on surface characteristics (roughness) of the aggregate particles, on the degree of bleeding, on chemical bonding (in some cases), and on the particular specimen preparation technique and is notoriously difficult to measure; it is not possible to reconcile the results of the various investigations that have been reported. However, it is generally found that, as the paste–aggregate bond strength increases, the concrete strength also increases, whether in tension, flexure, or compression. These increases range from about 5 to 40%, with improvements in tensile strength being higher than those in compressive strength.

In ordinary concretes, the ITZ has less crack resistance than either the aggregate or the hydrated cement paste, and so fracture occurs preferentially in the ITZ, which then constitutes the "weak link" in the concrete. However, to produce truly high-strength concretes [>70 MPa (10,000 lb/in.^{2})], it is necessary to increase the strengths both of the paste matrix and of the paste–aggregate bond. Unfortunately, reducing the *w/c* ratio has a much greater effect on the matrix strength than it does on the strength of the paste–aggregate bond. Thus, the focus has been on improving the strength of the ITZ itself, since fracture usually occurs not right at the physical interface, but about 10–20 μm into the ITZ. The most effective way of improving the ITZ is by the addition of 10–15% of silica fume by weight of cement. Silica fume (see Chapter 5) is very good in this regard for several reasons:

- It eliminates many of the larger pores in the ITZ, making its structure more homogeneous.
- It eliminates the growth of calcium hydroxide or transforms the calcium hydroxide to C–S–H by the pozzolanic reaction.
- It has a *filler effect* that modifies the rheological properties of the fresh concrete in such a way as to reduce internal bleeding, thus enhancing the paste–aggregate bond.

Currently, the use of silica fume is the most effective way of densifying the ITZ. However, other techniques are being examined, such as using chemical reagents, for example, surfactants or water glass in the concrete mix. Such treatments may, in the end, provide a more economical means of achieving better bonding.

For ordinary concretes, improvements in the ITZ (or in the bond strength) are unlikely to lead to major changes in concrete behavior. The modest increases in strength attributable to better paste–aggregate bonding are largely offset by the increasing brittleness of the resulting materials. However, for *high-performance* systems, such as very high-strength [>100 MPa (15,000 lb/in.^{2})] concretes, high-strength fiber-reinforced systems (Chapter 22), or systems containing special aggregates such as graded stainless steel, high bond strengths resulting from improvements in the properties of the ITZ appear to be more important if the full potential of these special materials is to be attained.

13.2 MODULUS OF ELASTICITY

Concrete is a nonlinear inelastic material in both tension and compression; therefore, the term *modulus of elasticity* must be applied with some caution since it does not represent a single value as it does for a linear elastic material. Rather, as shown in Figure 13.3, the term is adapted to describe the response of the material at various points along the

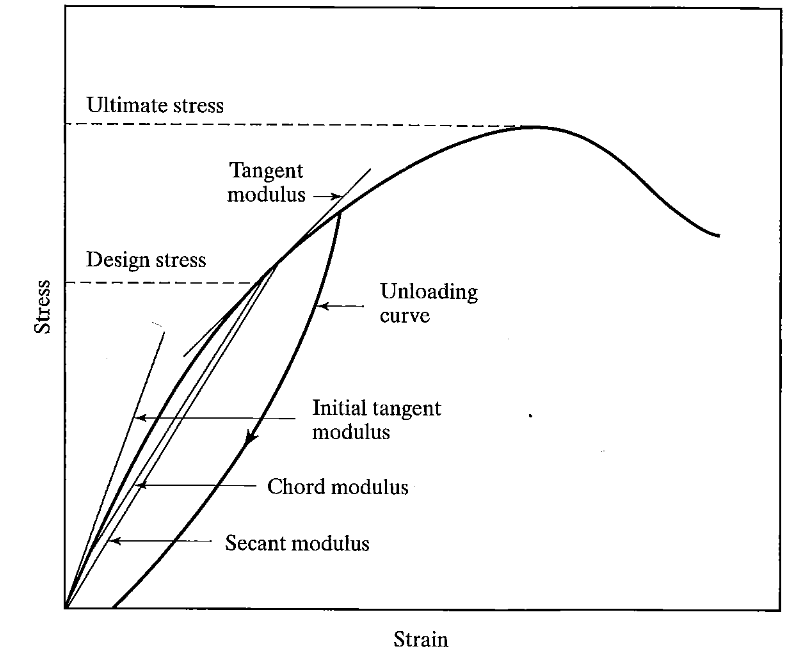


FIGURE 13.3

Typical stress–strain diagram for concrete, showing the different elastic moduli.

stress–strain curve. The closest approximation to a modulus of elasticity derived from a truly elastic response is the *initial tangent modulus*, which is the slope of the tangent to the curve at the origin. This modulus is not used in the design of concrete structures, since it applies only to small stresses and strains. It is likely that the dynamic modulus of elasticity, determined from the fundamental frequency of vibration of a specimen (Chapter 14), may be a reasonable estimate of the initial tangent modulus, since its determination involves only very small displacements of the material. A more practical measure of the modulus of elasticity is the *secant modulus*, which is equal to the slope of the secant between the origin and a point on the stress–strain curve. The secant modulus thus includes an element of nonlinearity, and clearly its value depends on the value of the level of applied stress chosen. The secant modulus is often used in design since it simplifies the calculation of section properties. The use of the secant modulus has little effect up to typical working stresses since the deviation from linear behavior is relatively minor within this stress range.

The initial tangent or the secant moduli are not always easily determined because of errors in measurement due to specimen seating at the initiation of a test or due to the existence of preexisting cracks that close up under applied load. In such a case, the *chord* modulus (ASTM C 469, Chapter 14) is used, which is the slope of a line drawn between two points on the stress–strain curve. The initial tangent modulus typically corresponds to the slope of the curve at a strain of 0.00005. This strain is used as the lower limit for the chord modulus to avoid any error in the initial portion of the stress–strain curve. The chord modulus and the secant modulus determined at 40% of the compressive strength

will not be very different, although the former will be more precise. The chord modulus is a more conservative measure than the initial tangent modulus and is more easily determined experimentally. On the other hand, the chord modulus will underestimate the additional strain that occurs when stress in excess of 40% of the compressive strength is imposed on the concrete. In such a case, the tangent modulus measured at the point of interest is a better measure of the concrete response to relatively small additional stresses.

Response as a Composite Material

Describing the modulus of elasticity of concrete as a function of the properties of its constituent materials provides a tool to better understand the nature of the response of the material to stress. Composite models are often used to determine the relative importance of these components in determining the overall behavior of concrete. To gain a basic understanding, concrete is often modeled as a two-phase material, with aggregate particles embedded in a matrix of cement paste, or as a three-phase material in which the third phase represents the interfacial transition zone (see Section 13.1). For the purposes of these models, the aggregate and paste phases are normally considered to be homogeneous and isotropic. Experience has shown that models of this type can provide very good approximations of the mechanical behavior of concrete if the properties of the components are known.

To fully describe a system with one or more dispersed phases (particles) embedded in a continuous matrix, it is necessary to consider the shape, size distribution, concentration, orientation, spatial distribution, and composition of the dispersed phase, the composition of the continuous phase, and the properties of the interfacial region between the continuous and dispersed phases.

Simply describing the geometry of a system such as concrete in terms of these parameters is difficult. Therefore, it is helpful to consider two extreme cases of phase arrangement. The parallel (or Voigt) and series (or Reuss) systems are shown in Figures 13.4a and 13.4b. The parallel system, in which the two phases are subject to uniform strains, provides the upper-bound solution for the elastic parameter of interest. The series system, in which the phases are subject to uniform stresses, provides the lower bound solution. Using these two models to calculate the modulus of elasticity results in the first two equations shown in Table 13.1, in which E_c, E_p, and E_a represent the moduli of elasticity of concrete, cement paste, and aggregate, respectively, and V_p and V_a are the volume fractions of paste and aggregate, respectively.

Neither model, of course, is quite correct since the components of concrete are subjected to neither uniform stress nor uniform strain under load (see Figure 13.24 in Section 13.4). For concretes made with natural aggregates, the uniform stress model underestimates E_c by about 10%, while the uniform strain model overestimates E_c by a greater amount. Other models have been proposed to provide a better representation of concrete.

One model, suggested by Hirsch, is obtained by combining the moduli obtained from the parallel and series models. The Hirsch model is illustrated by Figure 13.4c and expressed in the third equation in Table 13.1, where x and $(1 - x)$ are the relative proportions of material conforming to the upper and lower bound solutions, respectively. For many concretes, x is approximately 0.5.

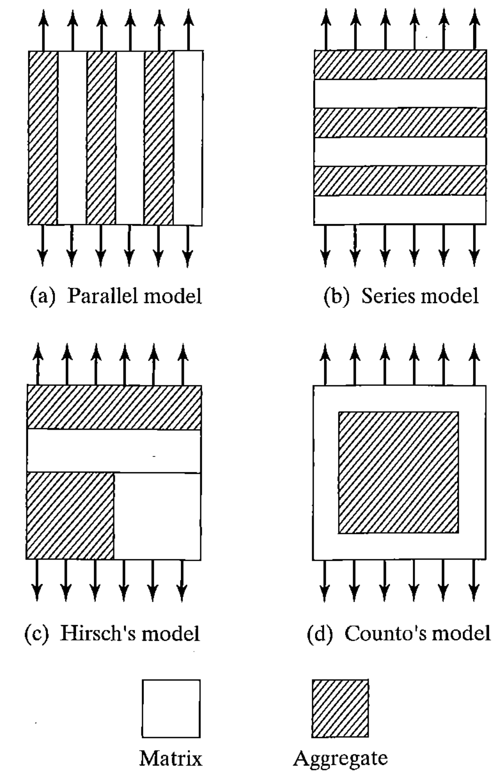


FIGURE 13.4

Models of the structure of concrete: (a) parallel (Voigt, constant-strain) model; (b) series (Reuss, constant-stress) model; (c) Hirsch's model [from T. J. Hirsch, *Journal of the American Concrete Institute*, Vol. 59, No. 3, p. 427 (1962)]; (d) Counto's model [from V. J. Counto, *Magazine of Concrete Research*, Vol. 16, No. 48, p. 129–138 (1964), reproduced by permission of British Cement Association, formerly Cement and Concrete Association].

One limitation of both the series and the Hirsch models is that for $E_a = 0$, they predict $E_c = 0$, which is clearly not true. Therefore, still another model, shown in Figure 13.4d and Table 13.1, was proposed by Counto. This model is compared graphically with the three other models in Figure 13.5.

An even more sophisticated representation of two-phase materials is provided by the Hashin–Shtrikman (H–S) bounds. As shown in Table 13.1, H–S bounds are expressed in terms of the bulk moduli K and shear moduli G of the component materials. The relationships between E, G, K, and Poisson's ratio ν are also given in Table 13.1. As shown in Figure 13.5, like the Hirsch and Counto models, the Hashin–Shtrikman bounds lie between the upper and lower bounds obtained with the parallel and series models. Studies have shown that moduli of elasticity calculated using the Hashin–Shtrikman bounds, as well as values of E_c calculated using the Hirsch and Counto models, provide good representations for many, but not all, concretes.

Differences between measured values of E_c and values predicted using these models (especially measured values below the lower H–S bounds) have been interpreted as being due to the presence of the interfacial transition zone, which acts as a third phase. Two models that can be used to calculate the elastic moduli of three-phase composites are shown in Table 13.2. The simpler of the two, the logarithmic mixture rule, requires knowledge of E_i and V_i, the modulus of elasticity, and volume fraction, respectively, of the ITZ. The more sophisticated model, the Hashin bounds, requires knowledge of K_i and G_i, the bulk and shear moduli of the interfacial transition zone,

TABLE 13.1 Elastic Moduli for Two-Phase Composite Materials

Parallel (Voigt) Model

$$E_c = V_p E_p + V_a E_a$$

Series (Reuss) Model

$$\frac{1}{E_c} = \frac{V_p}{E_p} + \frac{V_a}{E_a}$$

Hirsch Model

$$\frac{1}{E_c} = x\frac{1}{(V_p E_p + V_a E_a)} + (1 - x)\left(\frac{V_p}{E_p} + \frac{V_a}{E_a}\right)$$

Counto Model

$$\frac{1}{E_c} = \frac{1 - \sqrt{V_a}}{E_p} + \frac{\sqrt{V_a}}{(1 - \sqrt{V_a})E_p + \sqrt{V_a}\,E_a}$$

Hashin–Shtrikman Bounds

$$K = K_m + \frac{V_n}{\dfrac{1}{K_n - K_m} + \dfrac{3V_m}{3K_m + 4G_m}}$$

$$G = G_m + \frac{V_n}{\dfrac{1}{G_n - G_m} + \dfrac{6(K_m + 2G_m)V_m}{5G_m(3K_m + 4G_m)}}$$

To calculate upper bounds, $K_{(+)}$ and $G_{(+)}$, $m = 2$ and $n = 1$; to calculate lower bounds, $K_{(-)}$ and $G_{(-)}$, $m = 1$ and $n = 2$; where $K_2 = \max(K_p, K_a)$, $K_1 = \min(K_p, K_a)$; $G_2 = \max(G_p, G_a)$; $G_1 = \min(G_p, G_a)$.

Relationships between modulus of elasticity, Poisson's ratio, bulk modulus, and shear modulus

$$E = \frac{9KG}{3K + G} = 2G(1 + \nu) = 3K(1 - 2\nu)$$

$$\frac{9K_{(-)}G_{(-)}}{3K_{(-)} + G_{(-)}} \leq E_c \leq \frac{9K_{(+)}G_{(+)}}{3K_{(+)} + G_{(+)}}$$

$$\frac{3K_{(-)} - 2G_{(+)}}{6K_{(-)} + 2G_{(+)}} \leq \nu_c \leq \frac{3K_{(+)} - 2G_{(-)}}{6K_{(+)} + 2G_{(-)}}$$

and t_r, the ratio of the thickness of the ITZ to the equivalent radius of the (assumed) spherical inclusions, which represent the aggregate particles. Based on measurements of the dynamic modulus of elasticity (see Chapter 14) and an analysis using the logarithmic mixture rule, the ratio of E_i to E_p has been found to lie between 0.15 and 0.74, depending on the assumed thickness of the ITZ. For a similar concrete containing silica fume, the ratio lies between 0.50 and 0.88, demonstrating greater density of the ITZ when silica fume is used in concrete. The same study found the ratio G_i/G_p to lie between 0.09 and 0.65 for concrete without silica fume and between 0.30 and 0.81 for concrete with silica fume. Other studies have indicated that the ratios E_i/E_p and G_i/G_p lie between 0.5 and 0.7.

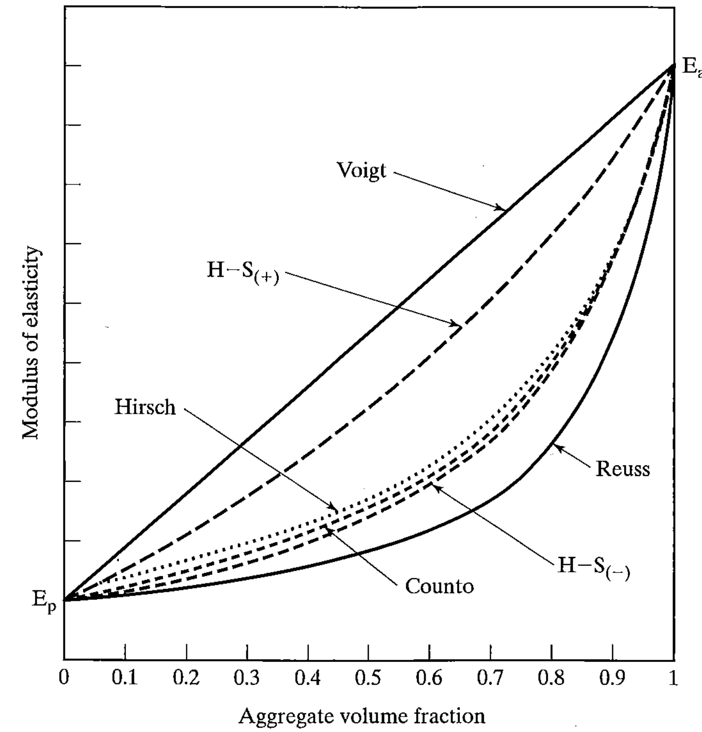


FIGURE 13.5

Relations between the modulus of elasticity of concrete and the volume fraction of aggregate for the structural models in Table 13.1. ($x = 0.5$ for Hirsch Model and $\nu = 0.2$ for Hashin–Shtrikman Bounds.)

TABLE 13.2 Elastic Moduli for Three-Phase Composite Material

Logarithmic Mixture Rule

$$\log E_c = V_p \log E_p + V_a \log E_a + V_i \log E_i$$

$$\text{or } E_c = E_p^{V_p} E_a^{V_a} E_i^{V_i}$$

Hashin Bounds

$$\frac{1}{K_{(-)}} = \frac{V_p}{K_p} + \frac{V_a}{K_a} + \frac{3V_a t_r}{K_i + 4G_i/3}$$

$$K_{(+)} = V_p K_p + \frac{V_a K_a}{1 + \dfrac{3K_a t_r}{K_i + 4G_i/3}}$$

$$\frac{1}{G_{(-)}} = \frac{V_p}{G_p} + \frac{V_a}{G_a} + 0.4V_a t_r\left(\frac{2}{K_i + 4G_i/3} + \frac{6}{G_i}\right)$$

$$G_{(+)} = V_p G_p + \frac{V_a G_a}{1 + \dfrac{2.5G_a t_r}{(K_i + 4G_i/3) + 2G_i}}$$

t_r = ratio of interface thickness to equivalent radius of spherical inclusions. See Table 13.1 for relationships between modulus of elasticity, Poisson's ratio, bulk modulus, and shear modulus.

Practical Predictions of Moduli

The material moduli calculated using the expressions in Tables 13.1 and 13.2 are used principally to gain a better understanding of the behavior of concrete as a material. For structural design, the modulus of elasticity must be estimated based on readily available information, such as concrete strength and density. Based on a large number of tests run using the procedures described in Chapter 14, empirical relationships between these properties and the modulus of elasticity have been established. In the ACI Building Code (ACI 318), the relationship used is

$$E_c = 0.043\, w_c^{1.5} \sqrt{f_c'} \quad \text{MPa} \tag{13.1a}$$

and

$$E_c = 33\, w_c^{1.5} \sqrt{f_c'} \quad \text{lb/in.}^2 \tag{13.1b}$$

where E_c is the secant modulus of elasticity (at about 45% of the ultimate strength), w_c is the unit weight of the concrete in kg/m^3 (lb/ft^3), and f_c' is the compressive strength of a standard 150 × 300 mm (6 × 12 in.) cylinder in (MPa lb/in.^{2}). Assuming a density for normal weight concrete of 2320 kg/m^3 (145 lb/ft^3), this reduces to

$$E_c = 4730 \sqrt{f_c'} \quad \text{GPa} \tag{13.2a}$$

$$E_c = 57{,}000 \sqrt{f_c'} \quad \text{lb/in.}^2 \tag{13.2b}$$

Additional expressions for E_c, principally for high-strength concretes, are given in Chapter 19.

Poisson's ratio, ν is not often required for design, but can be determined conveniently by direct strain measurements in uniaxial compression (ASTM C 469) or can be determined dynamically using Eq. (14.11) (Section 14.4). Poisson's ratio for saturated concrete lies in the range 0.2 to 0.3, but averages 0.18 upon drying. This change is not completely reversible on resaturation. For different concretes, Poisson's ratio generally falls in the range of 0.15 to 0.2, but the variation as a function of concrete properties is not known precisely. When determined dynamically, the value is somewhat higher, averaging about 0.24, and this is probably more representative of elastic behavior.

Factors Affecting the Modulus of Elasticity

As demonstrated empirically in Eq. (13.1), the modulus of elasticity is related to compressive strength and density. Thus, factors that affect strength should similarly influence modulus, and this is by and large true. The dominant factor is, of course, the porosity, and the modulus will decrease markedly as the *w/c* ratio is increased. It should be remembered that, since the porosity changes that occur with *w/c* are wholly within the paste, this change in the stress–strain behavior of concrete is due to changes in the response of the paste.

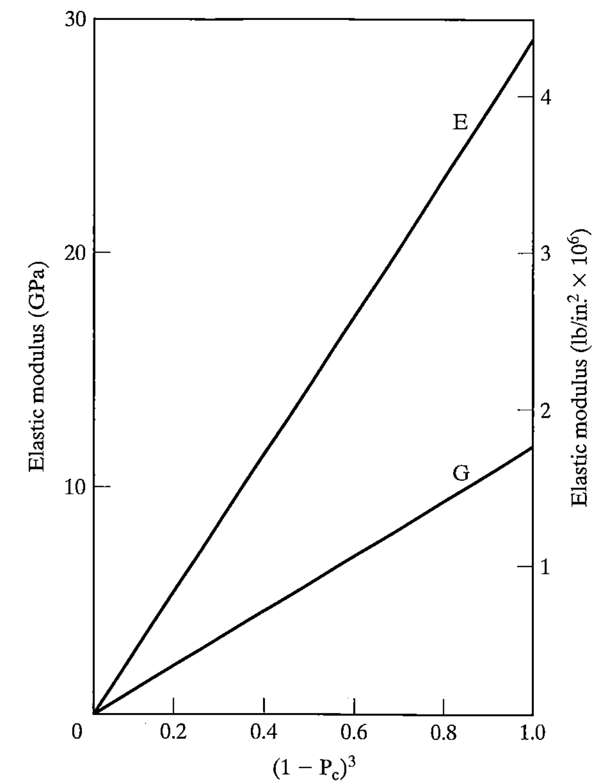


FIGURE 13.6

Dependency of elastic moduli on the porosity of cement paste. [From G. J. Verbeck and R. A. Helmuth, *Proceedings, Fifth International Symposium on the Chemistry of Cement*, Tokyo, Vol. 3, pp. 1–32 (1968).]

The modulus of elasticity of cement paste, E_p, has been found to be approximately proportional to the cube of the gel/space ratio (see Chapter 4). In terms of capillary porosity, the relationship can be written as

$$E_p = E_g(1 - P_c)^3 \tag{13.3}$$

where E_g is the modulus of elasticity of the hardened cement paste at zero porosity and P_c is the capillary porosity expressed as a ratio. A similar relationship holds for the shear modulus, G (Figure 13.6). For ordinary portland cement, E_g appears to be about 29 GPa (4.2 × 10^6 lb/in.^{2}), while G_g is about 12 GPa (1.75 × 10^6 lb/in.^{2}).

One apparent inconsistency in the strength–modulus relationship is the moisture dependency of the two properties. The strength of saturated concrete is lower than that of dry concrete, while for the elastic modulus, the reverse is true. The modulus of elasticity of concrete, however, is much more sensitive to the amount and nature of the aggregate than it is to moisture content, as shown in Figure 13.7. It is worth noting that the relative effects of the two conventional aggregates shown in the figure—gravel and limestone—should not be construed as general, since this comparison applies only to the particular aggregates used to develop the data shown (that is, many limestones will

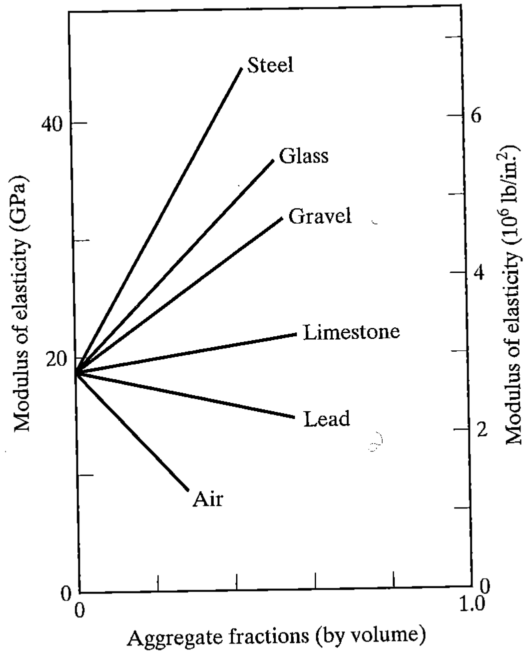


FIGURE 13.7

Effect of aggregates on the modulus of elasticity of concrete. [Adapted from O. Ishai, *Journal of the American Concrete Institute*, Vol. 59, No. 9, pp. 1365–1368 (1961).]

provide stiffer concretes than those obtained with many types of gravel). The much lower elastic modulus of porous, lightweight aggregates, such as expanded clay or shale, has a strong influence on the elastic modulus of concrete. E_c of lightweight concrete is 40 to 80% that of normal-weight concrete (see Table 13.3). The stress–strain behavior of lightweight concrete is nearly linear over a much wider stress range than it is for normal-weight concrete. Equation (13.1) applies to lightweight as well as normal-weight concrete. As discussed in Chapter 19, however, more accurate expressions are available for concretes with strengths above 40 MPa (6000 lb/in.^{2}). The stress–strain curve of concrete, and hence the modulus of elasticity, is dependent on the conditions of testing. An important parameter is the applied strain rate (Figure 13.8). As discussed in Chapter 16, the higher the strain rate, the greater the strength and the higher the modulus of elasticity. Fortunately, in the time range normally required to test a specimen (2 to 10 min), the effect of strain rate is very small. The sensitivity of E_c to testing conditions is also demonstrated by the dynamic modulus of elasticity. In the dynamic test, concrete is subjected to very small strains; therefore, its response is more closely related to

TABLE 13.3 Moduli of Elasticity for Concrete and Its Components, GPa (lb/in.^{2})

	Normal-Weight	*Lightweight*
Aggregate	70–140 (10–20 × 10^6)	14–35 (2–5 × 10^6)
Cement paste	7–28 (1–4 × 10^6)	7–28 (1–4 × 10^6)
Concrete	14–42 (2–6 × 10^6)	10–18 (1.5–2.5 × 10^6)

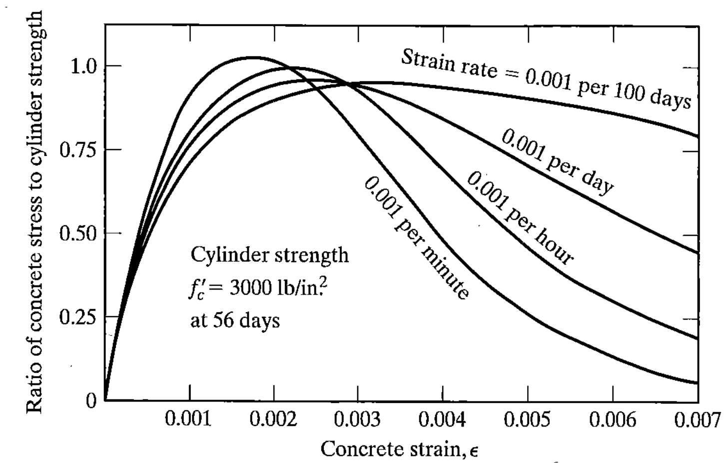


FIGURE 13.8

Stress–strain curves for various strain rates under compressive loading. [Adapted from H. Rusch, *Journal of the American Concrete Institute*, Vol. 57, No. 1, pp. 1–23 (1960).]

the initial tangent modulus than to the chord modulus. As a result, the dynamic elastic modulus is 20 to 30% higher than its static counterpart, depending on the strength of the concrete. The dynamic elastic modulus is used primarily to evaluate soundness of concrete in durability tests (Chapter 18) or for in-situ evaluations. However, it may be a more appropriate value when the concrete is to be used in structures that will be subjected to dynamic loadings (i.e., earthquake or impact).

The shear modulus G, which governs behavior under shear stresses, is not easily determined from direct stress–strain measurements. It is probably best determined by dynamic methods using the resonant frequency of torsional vibration of a concrete specimen (see Chapter 14).

13.3 TENSION AND FRACTURE

The tensile strength of concrete is much lower than the compressive strength, because of the ease with which cracks can propagate under tensile loads. The tensile strength of concrete is usually not considered in design (it is often assumed to be zero). However, it is an important property, since cracking in concrete is most generally due to the tensile stresses that occur under load, or due to environmental changes. The failure of concrete in tension is governed by microcracking, associated particularly with the interfacial region between the cement and the aggregate particles (the ITZ, Section 13.1).

Tensile Strength

As will be discussed in Chapter 14, the tensile strength of concrete is most often evaluated using a flexure test, in which plain concrete is loaded in bending, or the split cylinder test, in which a cylindrical specimen is placed on its side and loaded in diametral compression

so as to induce transverse tension. In North America, there is no standard test procedure to measure the direct tensile strength of concrete; a number of specimens (none yet standard) are available to measure the fracture properties of concrete under tension.

As a tensile crack propagates through concrete, its leading edge often consists of multiple branching microcracks that eventually coalesce into a single macrocrack as the tensile displacement increases. Depending on the relative tensile strength of the mortar, the aggregate, and the ITZ, the macroscopic appearance of the failure surface may be smooth or rough. A low ratio of aggregate to ITZ strength will result in a relatively smooth failure surface. As the relative strength of the aggregate increases, the failure surface becomes progressively more uneven, usually resulting in a higher tensile strength and improved fracture properties. Because compressive strength is the principal material property that is measured for hardened concrete, the relationship between tensile and compressive strength is of particular interest.

In general, as age and strength increase, the ratio of tensile to compressive strength (f_t'/f_c') decreases, as shown in Figures 13.9 and 13.10. Also, since crushed coarse aggregate seems to improve tensile strength more than it does compressive

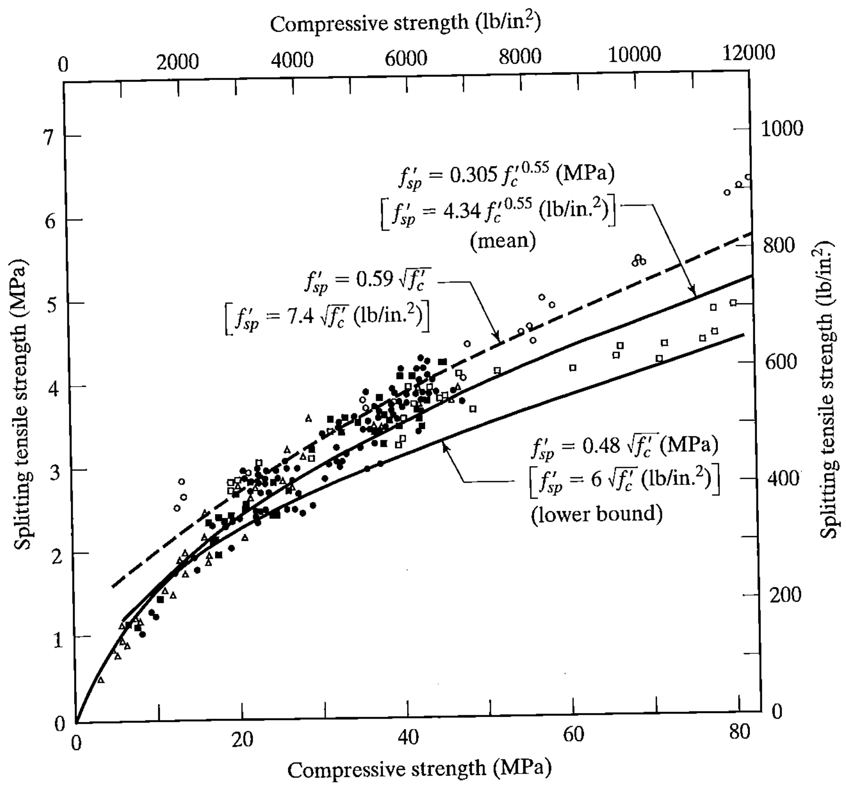


FIGURE 13.9

Split cylinder tensile strength versus compressive strength for normal-weight concretes. [Adapted from S. H. Ahmad and S. P. Shah, *PCI Journal*, Vol. 30, No. 6, pp. 92–119 (1985).]

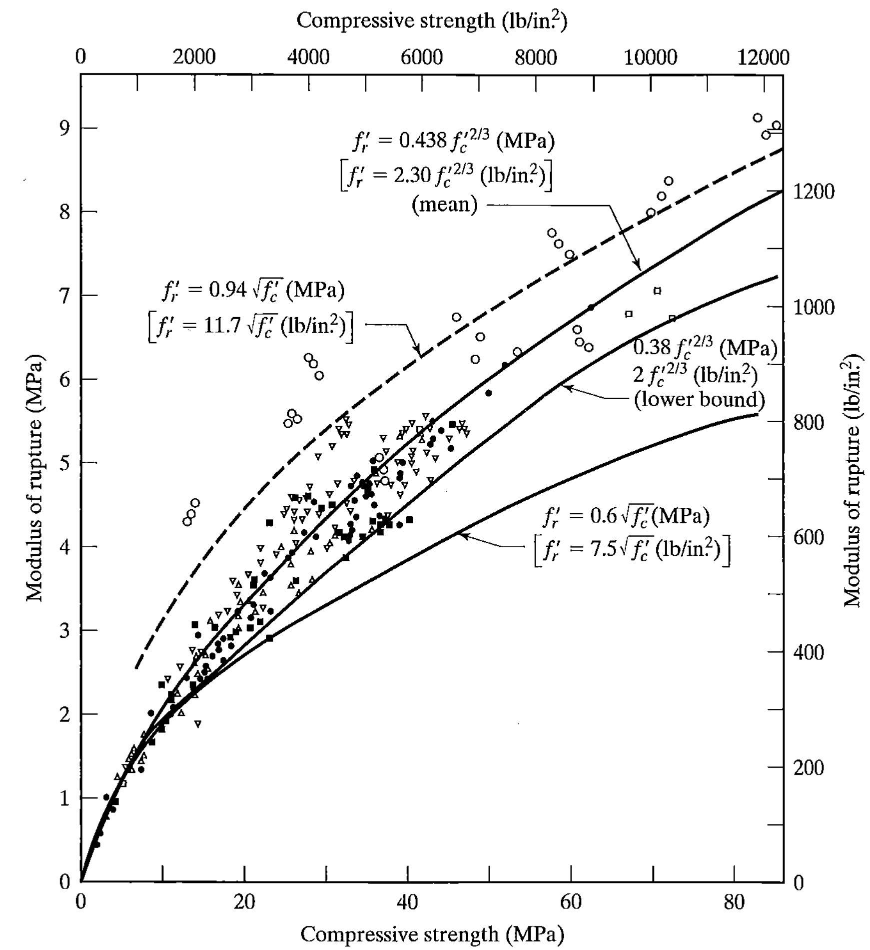


FIGURE 13.10

Modulus of rupture versus compressive strength for normal-weight concretes. [Adapted from S. H. Ahmad and S. P. Shah, *PCI Journal*, Vol. 30, No. 6, pp. 92–119 (1985).]

strength, the f_t'/f_c' ratio also depends on the type of aggregate. It has been found that, compared to moist curing, air curing reduces the tensile strength more than it does compressive strength, probably because of the effect of drying shrinkage cracks. Thus, the f_t'/f_c' ratio is lower for air-cured than it is for moist-cured concretes. Incomplete consolidation and air entrainment, on the other hand, affect the compressive strength more than they do the tensile strength.

In general, the ratio of direct tensile strength to compressive strength ranges from about 0.07 to 0.11. Since splitting tension (f_{sp}') values tend to be slightly higher than direct tension values, the f_{sp}'/f_c' ratios are also slightly higher, from about 0.08 to 0.14. Flexural tests give results that are substantially higher than direct tension tests. The ratio of flexural strength (or *modulus of rupture*, f_r') to compressive strength (f_r'/f_c') ranges from about 0.11 to 0.23.

A number of expressions have been developed to represent the relationship between flexural and compressive strengths. The relationship that is used by ACI is

$$f_r' = 0.6\sqrt{f_c'} \quad \text{MPa} \tag{13.4a}$$

$$f_r' = 7.5\sqrt{f_c'} \quad \text{lb/in.}^2 \tag{13.4b}$$

Analysis of a wide range of data, however, indicates that best-fit expressions for both the splitting tension and flexural strength of concrete depend on a power of f_c' greater than the square root.

$$f_{sp}' = 0.305 f_c'^{0.55} \quad \text{MPa} \tag{13.5a}$$

$$f_{sp}' = 4.34 f_c'^{0.55} \quad \text{lb/in.}^2 \tag{13.5b}$$

$$f_r' = 0.438 f_c'^{2/3} \quad \text{MPa} \tag{13.6a}$$

$$f_r' = 2.30 f_c'^{2/3} \quad \text{lb/in.}^2 \tag{13.6b}$$

The relationships in Eqs. (13.4)–(13.6), along with several others, are illustrated in Figures 13.9 and 13.10. As shown in the figures, the actual relationships between tensile and compressive strengths vary widely and exhibit significant scatter.

To understand the nature of the tensile behavior of concrete, it is necessary to understand the fracture mechanisms. However, before discussing these in detail, it is necessary first to give a brief account of the science of fracture mechanics.

Linear Elastic Fracture Mechanics

Linear elastic fracture mechanics, or LEFM, may be defined as the study of the stress and displacement fields in the region of a crack tip in materials that are elastic, homogeneous, and isotropic, particularly at the onset of unstable crack growth (or fracture). This concept is applicable to brittle materials in which inelastic behavior is at a minimum. We will discuss classical fracture mechanics and then see to what extent these concepts apply to a composite such as concrete.

Theoretical Cohesive Stress Ideally, the strength of a solid depends on the strength of its atomic bonds. Thus, to obtain an approximation to at least an order of magnitude of the fracture strength, we can consider the interaction between two atoms. Figure 13.11 a shows a typical curve representing the energy of interaction of these two atoms as a function of their separation. This energy is a minimum at the equilibrium atomic spacing, b_0. The total energy required to separate these atoms completely is U_0. However, since one of the consequences of fracture is the creation of new surface areas along the fracture plane, this work to fracture may be equated to twice the surface energy, γ_s, of the new surface area created, as two new surfaces are created when a bond is broken.

If we differentiate the energy–distance curve, we can obtain the force–displacement curve shown in Figure 13.11b. The force is zero at the equilibrium atomic spacing; the initial slope of this curve is proportional to the modulus of elasticity, E. In practice, the force–displacement curve can be transformed to a stress–strain curve that can be approximated by a half sine wave, as shown in Figure 13.12. The area under this curve represents the work to bring about fracture, $2\gamma_s$; the initial slope is E. The relationship

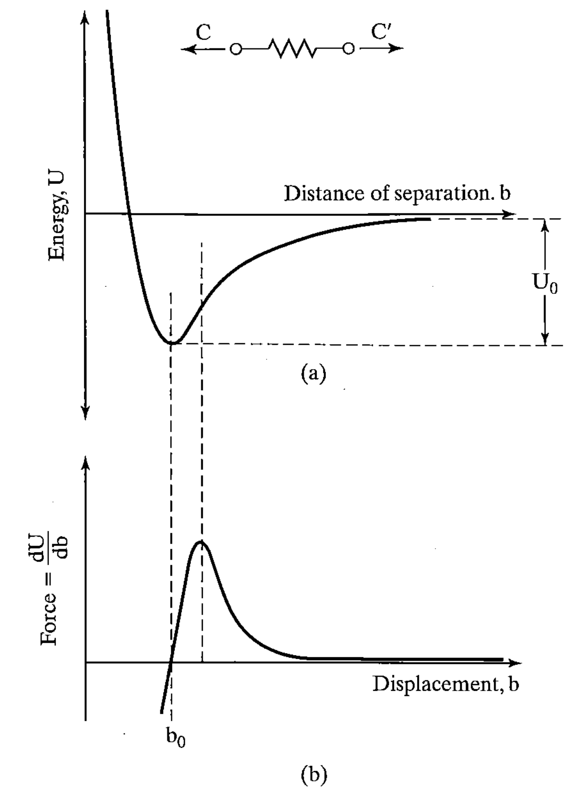


FIGURE 13.11
(a) Bonding energy as a function of distance of separation; (b) force–displacement curve.

between stress, σ, and the displacement, x, is

$$\sigma = \sigma_{max} \sin\frac{2\pi x}{\lambda} \tag{13.7}$$

Thus, the work to fracture can be written as

$$2\gamma_s = \int_0^{\lambda/2} \sigma_{max} \sin\frac{2\pi x}{\lambda}\,dx = \frac{\lambda\sigma_{max}}{\pi} \tag{13.8}$$

But for small displacements, Hooke's law is obeyed, and

$$\sigma = E\epsilon = E\frac{x}{b_0} \tag{13.9}$$

From Eq. (13.7), since for small angles $\sin x \approx x$,

$$\sigma_{max} = \frac{\lambda E}{2\pi b_0} \tag{13.10}$$

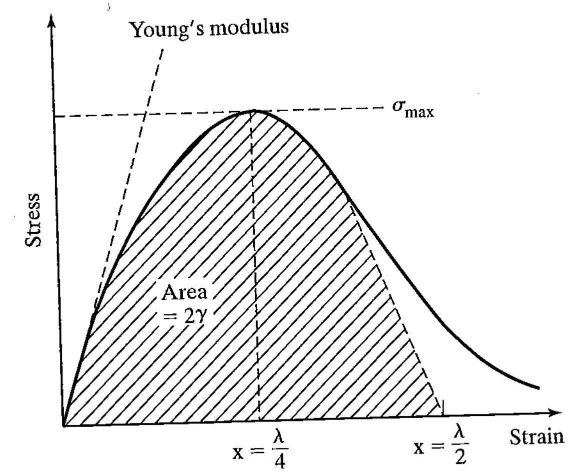


FIGURE 13.12
Atomic stress–strain curve.

So, equating Eqs. (13.8) and (13.10), we get

$$\sigma_{max} = \left(\frac{E\gamma_s}{b_0}\right)^{1/2} \tag{13.11}$$

where σ_{max} is the theoretical cohesive strength. For typical values of γ_s and b_0, a reasonable estimate of the theoretical cohesive strength for solids is of the order of $E/10$. For concrete, this would imply a theoretical strength of about 2100 MPa (300,000 lb/in.^{2}).

Griffith Theory Based on thermodynamic considerations, Griffith arrived at a similar solution of the theoretical cohesive strength. Considering an elastic body containing a crack and subjected to external loads, he calculated the condition at which the total free energy of the system was minimized. The total energy in the system is

$$U = (-W_L + U_E) + U_s \tag{13.12}$$

where W_L = work due to the applied loads, U_E = strain energy stored in the system, and U_s = free surface energy in creating new crack surface. Then a crack would propagate when $dU/dc < 0$. Using this theory, we can derive the Griffith equation for the theoretical fracture strength,

$$\sigma_f = \left(\frac{2E\gamma_s}{\pi C}\right)^{1/2} \tag{13.13}$$

where C is one-half the crack length. At σ_f, $C \approx b_0$. This is very similar to Eq. (13.11), even though it was derived in quite a different fashion.

The question then arises: Why is there this enormous discrepancy, perhaps two or three orders of magnitude, between theoretical values of strength [as predicted by Eq. (13.11) or (13.13)] and those actually measured? The answer to this question was first suggested by Griffith in 1920, who concluded that any real material must contain flaws, microcracks, or other defects that have the effect of concentrating the stress sufficiently to

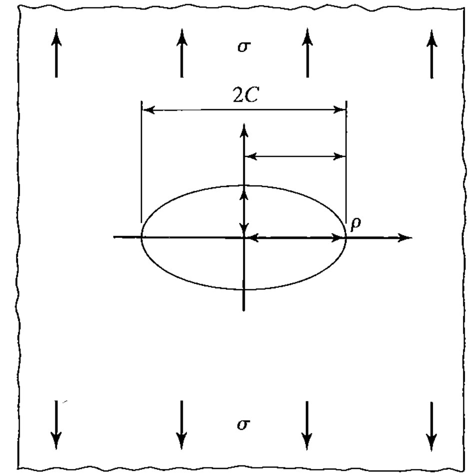


FIGURE 13.13
Elliptical hole in an infinite plate in tension.

reach the theoretical cohesive stress in highly localized regions of the specimen. Cracks then grow under an applied stress until failure occurs. It is easy enough to show that cracks can indeed concentrate the stress sufficiently to achieve these very high stresses locally. For a plate in uniform tension, containing an elliptical hole (which in the limit might represent a crack), as shown in Figure 13.13, then the stress at the crack tip can be written as

$$\sigma_{max} = \sigma\left(1 + 2\sqrt{\frac{C}{\rho}}\right)^{1/2} \tag{13.14a}$$

where ρ is the radius of the crack tip, or

$$\frac{\sigma_{max}}{\sigma} = 2\sqrt{\frac{C}{\rho}} \quad \text{for } C \gg \rho \tag{13.14b}$$

Clearly, for $C \gg \rho$, that is, for a very "sharp" crack, this factor can become very large. The form of the stress distribution ahead of the crack is shown in Figure 13.14.

Stress Intensity Factor. Although there are three modes of crack displacement (Figure 3.15), the crack opening mode (Mode I) is the most important one to consider for brittle materials. For the stress field near a sharp crack, as shown in Figure 13.16, the stresses can be written as

$$\begin{Bmatrix}\sigma_x \\ \sigma_y \\ \tau_{xy}\end{Bmatrix} = \frac{K_I}{(2\pi r)^{1/2}}\begin{Bmatrix}\cos\left(\frac{\theta}{2}\right)\left[1 - \sin\left(\frac{\theta}{2}\right)\sin\left(\frac{3\theta}{2}\right)\right] \\ \cos\left(\frac{\theta}{2}\right)\left[1 + \sin\left(\frac{\theta}{2}\right)\sin\left(\frac{3\theta}{2}\right)\right] \\ \sin\left(\frac{\theta}{2}\right)\cos\left(\frac{\theta}{2}\right)\cos\left(\frac{3\theta}{2}\right)\end{Bmatrix} \tag{13.15}$$

$$\sigma_z = \nu(\sigma_x + \sigma_y) \qquad \tau_{xz} = \tau_{yz} = 0$$

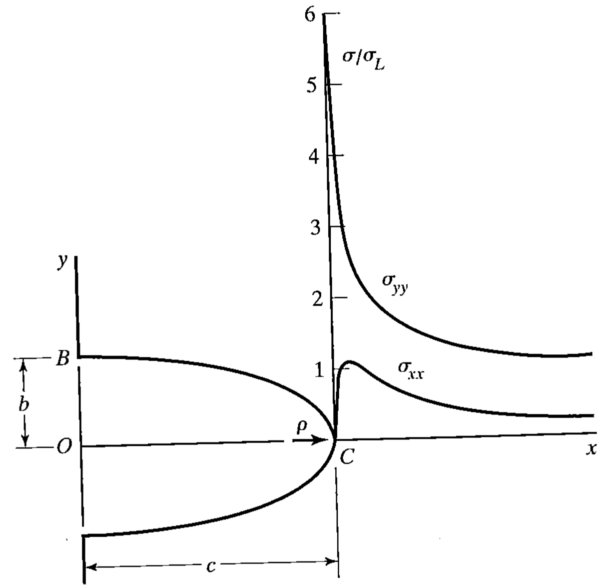


FIGURE 13.14

Stress concentration at elliptical hole, $c = 3b$. Note that concentration localized within $\approx c$ of the tip, with high stress gradients within $\approx \rho$ of the tip.

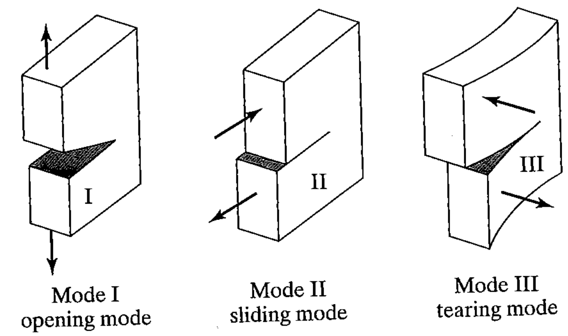


FIGURE 13.15

Three modes of cracking.

where ν is Poisson's ratio. The parameter K_I, the stress intensity factor, has the form $K_I = \sigma\sqrt{\pi C}$ for an infinite solid. For specimens with finite dimensions, this becomes $K_I = Y\sigma\sqrt{\pi C}$, where Y is a modification factor that takes into account the geometry of the specimen. K_I has the dimensions of stress $\times \sqrt{\text{length}}$, having units of $\text{N}\cdot\text{m}^{\frac{3}{2}}$ (or $\text{lb/in.}^{\frac{3}{2}}$), and may be considered to be a single-parameter description of the stress and displacement fields in the region of a crack tip. The underlying assumption is that when the stress intensity factor reaches some critical value, unstable fracture occurs. This *critical stress intensity factor* is designated K_{Ic} and is sometimes referred to as *fracture toughness*. It should be a fundamental material property, independent of how it is measured.

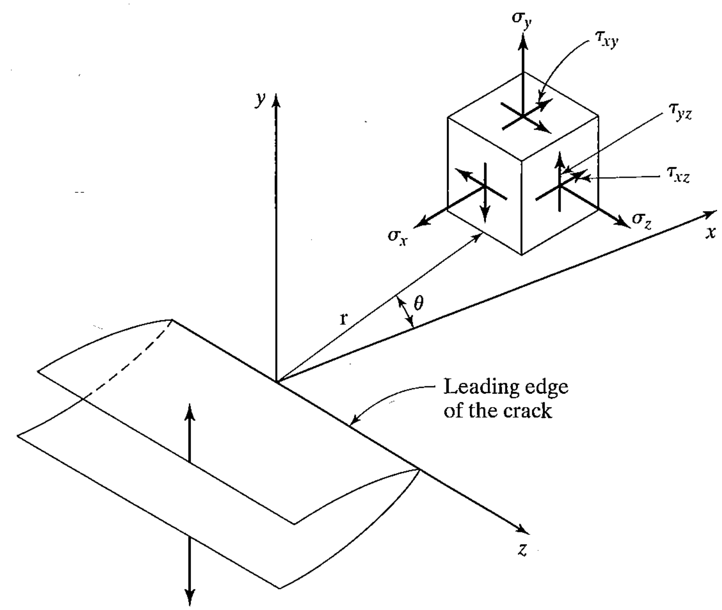


FIGURE 13.16

Coordinate system and stress components ahead of a crack tip (Mode I displacement).

An alternative way of considering fracture involves not K_I, but the *strain energy release rate*, G_I.[1] Unstable crack extension occurs when G_I reaches the critical strain–energy release rate, G_{Ic}. It can be shown that

$$G_{Ic} = \frac{K_{Ic}^2}{E} \qquad \text{plane stress}$$

or (13.16)

$$G_{Ic} = \frac{K_{Ic}^2}{E(1-\nu^2)} \qquad \text{plane strain}$$

so that these are equivalent ways of expressing the fracture criterion. It can also be shown that $G_{Ic} = 2\gamma_s$.

Now if the fracture criterion just described is applied to concrete, it should be possible to calculate γ_s for concrete, or at least for hardened cement paste. However, when this calculation is performed, values of γ_s that are at least an order of magnitude larger than the accepted value for the surface energy of cement paste are obtained.

[1]The terms K and G used in this section are distinct from and, in fact, not associated with the bulk and shear moduli (K and G) discussed in Section 13.2. The use of the same notation for these terms is an historical anomaly, but rarely causes difficulties in understanding.

This would imply that the simple fracture mechanics theory described cannot be used for cement or concrete.

There are a number of reasons for this discrepancy in γ_s values. The calculation of K_{Ic} (or G_{Ic}) assumes a linearly elastic material that is both homogeneous and isotropic. However, these assumptions are incorrect for concrete. (It should be noted that for *very large* sections, such as those found in concrete dams, where the section size is much greater than the maximum aggregate size, linear elastic fracture mechanics *can* be applied successfully.) As well, in concrete, the crack does not necessarily propagate in a straight line (as assumed by Griffith), but follows a tortuous path, around aggregate particles or perhaps around different phases in the cement paste itself. There is also often multiple cracking or branch cracking at the crack tip, and a diffuse zone of microcracking, referred to as the fracture process zone, that moves ahead of the crack tip. Therefore, it is now more common to use *nonlinear fracture parameters*, which provide a better description of the fracture process.

Nonlinear Fracture Mechanics

A detailed account of nonlinear fracture mechanics is beyond the scope of this book. However, some of the principal models will be described briefly here.

Cohesive Crack Models In *cohesive crack models*, it is assumed that a set of discrete forces, acting on the crack faces in such a way as to tend to close the crack, can be used to approximate the fracture zone. These forces decrease the magnitude of K_{Ic} or G_{Ic} at the crack tip, and thus tend to toughen the concrete. These forces must be related to the concrete material parameters in some prescribed way. They are also often used to represent the bridging actions of fibers (see Chapter 22) or aggregate particles across the crack tip. To provide a complete description of concrete behavior, the cohesive crack model must be combined with a model that describes the behavior of the concrete before it cracks; the uncracked concrete is generally assumed to be linearly elastic.

R-Curve Models For linearly elastic materials, the resistance of the material against fracture, R, is equivalent to the energy release rate, G, and is assumed to be a material constant, independent of specimen size or geometry. For nonlinear materials, this is no longer true: R increases as the crack length increases. It is then possible to develop R-curves for the materials, which are plots of R versus crack extension; these R-curves can then be used to characterize the material. For perfectly brittle material behavior, R is essentially constant as the crack extends. For materials such as concrete, R increases monotonically to a horizontal asymptote. This behavior suggests that some amount of stable fracture occurs before the specimen finally fails catastrophically. Unfortunately, while R-curves can be used for fracture analysis of structural elements, they are not true material constants, but depend on the geometry of the structural element.

Two-Parameter Fracture Model The two-parameter fracture model (TPFM) involves two independent parameters to characterize the material:

1. K_{Ic}, the critical stress intensity factor, and
2. $CTOD_c$, the critical crack tip opening displacement.

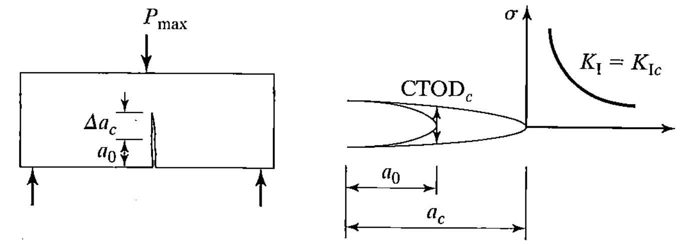


FIGURE 13.17
Two-parameter model fracture criteria for concrete.

For fracture to occur, two criteria must be satisfied simultaneously,

$$K_I = K_{Ic} \text{ and } CTOD = CTOD_c \tag{13.17}$$

as shown in Figure 13.17.

Fracture Energy Models The R-curve models and the two-parameter fracture model have the goal of predicting the onset of unstable crack growth. Other models have been developed with the goal of fully describing the response of brittle materials in tension. Principal among these models is the fracture energy model, in which the principal parameters are the tensile strength, $f_t' = \sigma_t'$, at which the material begins to lose its ability to sustain stress; the fracture energy, G_f, which is the total energy required to form a new, fully separated crack surface; and a shape factor that describes the change in stress with increasing displacement at the crack tip. Models of this type have been used successfully to represent the full nonlinear behavior of concrete in reinforced concrete members that are governed by the tensile or fracture properties of the material. As will be discussed in Chapter 19, fracture energy depends principally upon the strength of the coarse aggregate and is largely independent of concrete compressive strength, age, or *w/cm* ratio.

Other, more elaborate nonlinear fracture models have been developed, but they will not be discussed further here. There are, as yet, no standard fracture mechanics tests prescribed for concrete by ACI or ASTM, but such tests are in the process of development.

Fracture in Compression So far, we have only considered tensile stresses, while concrete is used primarily in compression. However, it should be realized that pure compression, which merely forces the molecules making up the structure of concrete closer together, cannot in itself cause "compressive failure." Rather, it is the *secondary tensile stresses*, induced by the application of the compressive force, that are actually responsible for the failure, as described in greater detail in Section 13.4. Nonetheless, the Griffith analysis can be extended to include uniaxial compression, as well as biaxial stress states. Griffith showed that under biaxial compressive stresses, the presence of small cracks leads to tensile stresses at some points along the edge of the flaw, as long as the stress components are unequal. Using the normal convention that $\sigma_1 > \sigma_2 > \sigma_3$, with tension positive, the Griffith criterion becomes

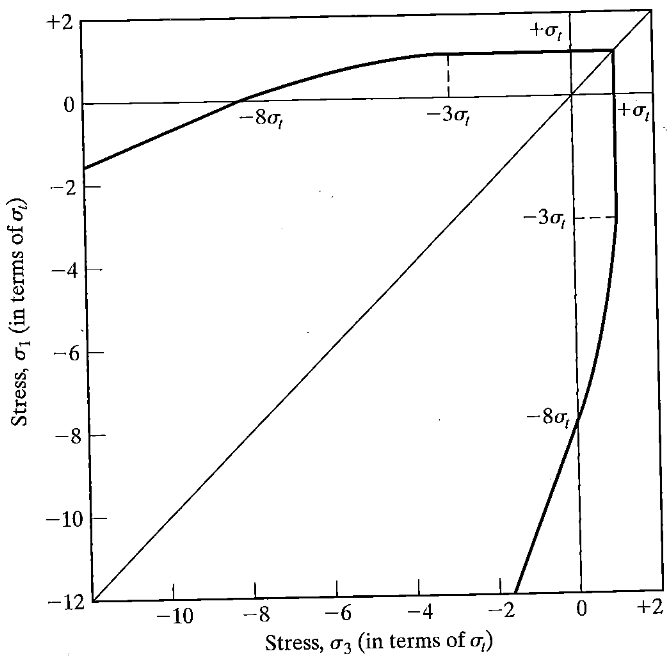


FIGURE 13.18

Griffith's criterion of fracture under biaxial stress (tension positive).

$$\sigma_1 = \sigma_t \quad \text{if } 3\sigma_1 + \sigma_3 > 0 \tag{13.18}$$

and

$$(\sigma_1 - \sigma_3)^2 + 8\sigma_t(\sigma_1 + \sigma_3) = 0 \quad \text{if } 3\sigma_1 + \sigma_3 < 0 \tag{13.19}$$

where σ_t is the uniaxial tensile strength. This is shown graphically in Figure 13.18. This criterion predicts that the compressive strength of concrete is eight times the tensile strength, which is fairly close to the observed values. This failure criterion agrees well with experimental data in the tension–compression zone, but does not work too well in the compression–compression zone, where splitting will generally occur normal to the unloaded σ_2 direction. Nevertheless, it is clear that concrete failure is due to the propagation of microcracks under localized tensile stresses, regardless of the type of loading (with the exception of hydrostatic compression).

On the other hand, it has also been suggested that failure is controlled not by this limiting tensile stress, but rather by a limiting tensile strain, on the order of 1 to 2×10^{-4}. This might then explain the failure of a compression cylinder by splitting, when the lateral strain ($= \nu \times$ longitudinal strain) reaches this limiting value. It may well be that the failure criterion for concrete should specify both a limiting strain and a limiting stress.

Difficulties in Application Fracture mechanics was first applied to concrete in 1959, and since then a number of studies have been carried out. However, as yet, no standard fracture tests or test specimens have been developed, so experimental results, are often contradictory and are certainly difficult to interpret. The most important problems have to do with specimen size and with the heterogeneity introduced by the aggregate. A notch-sensitive material is one in which the presence of a sharp notch reduces the

tensile or flexural strength beyond that caused by the mere reduction in cross-sectional area. In view of what we have just discussed, for fracture mechanics to apply, a material must be notch sensitive. It is generally found that cement paste, mortar, and concrete are all notch sensitive. One problem seems to be that for concrete, the zone of disturbance at the crack tip is large compared to the size of small specimens. It has been suggested, but not universally accepted, that a specimen depth of at least 250 mm (10 in.) is required if fracture studies on concrete are to be valid. The fracture process is further complicated by the aggregate, as the growth of a Griffith crack can be arrested by encountering an aggregate particle, or by crack-tip blunting due to debonding at the paste–aggregate interface.

Whatever the details of the failure mechanism, it appears that a nonlinear fracture mechanics approach is needed to characterize the fracture of concrete. However, there is still considerable uncertainty, not only of the values of the fracture parameters for different concretes, but even as to how these fracture parameters should be obtained.

Other failure criteria have been suggested, but none satisfactorily describes the failure of concrete. Most likely, the failure criterion for concrete should be an energy criterion of some sort, since the energy involved in cracking should be capable of definition. However, at present, it would appear that at least two criteria for failure should be considered, as pointed out by Newman[2]:

1. *a criterion for discontinuity* or the onset of failure when the concrete can no longer withstand the applied loading system without severe disruption of the structure;
2. *a criterion for strength* or the maximum loads that concrete can withstand under various stress combinations.

13.4 COMPRESSION

Because compressive strength is so much greater than tensile strength, compressive strength is widely considered to be the most important property of concrete. In this section, we discuss the details of the physical response of concrete to compressive stress, the resulting stress–strain curves, and the effects of testing system properties on that response. An understanding of the nature of the physical response and the impact of testing conditions is useful in both interpreting the results of compressive tests and understanding the behavior of concrete under load.

Nature of Response

The stress–strain behavior of concrete is normally evaluated using a uniaxial compression test in which a concrete cylinder or prism is loaded along a single axis. The standard test in North America, discussed at greater length in Chapter 14, uses a cylinder with a height-to-diameter ratio of 2, loaded between rigid test platens. Due to differences in material properties, the lateral strain due to Poisson's effect is greater for the

[2]K. Newman, in *The Structure of Concrete*, ed. A. E. Brooks and K. Newman, Cement and Concrete Association, London, pp. 255–274 (1968).

concrete than for the end platens, resulting in friction between the test specimen and the platens, which places the end regions of the specimen in triaxial compression. This results in the formation of conical regions at the ends of the specimen, as shown in Figure 14.2a (Section 14.3). Mild lubrication of the bearing surfaces can reduce the friction so that the true tensile nature of compressive failure, as discussed in the previous section, is attained, as shown in Figure 14.2c.

In addition to lubricating the contact surfaces between the test specimen and the testing machine, other procedures have been used to help attain true uniaxial states of stress. One procedure involves the use of brush bearing platens in which rigid platens are replaced by a number of small, individual column-like platens that can apply compressive stress, but which move laterally as the concrete expands. Such test systems have been widely used for basic material studies, but an analysis of displacements at the platen-specimen surface indicates that lateral displacement of the "brushes" results in a nonuniform application of compressive stress at the specimen surface (the stresses are highest at the center line of the specimens and drop off toward the edge). As shown in Figure 13.19, when concrete is subjected to a state of stress that is close to uniaxial (i.e., using brush bearing platens), the height-to-width ratio has little effect on the ascending portion of the stress–strain curve or the compressive strength, principally controlling the descending portion of the stress–strain curve and emphasizing the *structural* nature of specimen response.

A simpler method for attaining a uniform state of uniaxial stress involves an increase in the aspect ratio of the compression specimen so that the height is approximately three times the cross-sectional dimension. Finite element analyses indicate that the central region of such test specimens are subjected to a nearly uniform state of uniaxial compression.

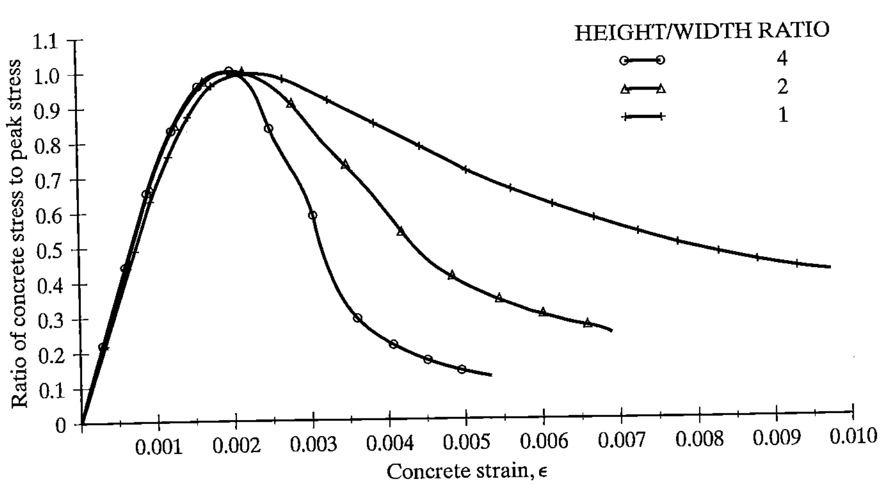


FIGURE 13.19

Influence of specimen aspect ratio on uniaxial stress–strain curves for concretes loaded with brush bearing platens. [Adapted from J. G. M. van Mier, *Strain Softening of Concrete Under Multiaxial Loading Conditions*, Eindhoven University of Technology, Eindhoven, The Netherlands (1984).]

Physical Nature of Response As described earlier in this chapter, concrete is a composite material consisting of an essentially linear material, aggregate, and a highly nonlinear material, cement paste, separated by an interfacial transition zone that, in most cases, behaves as a weak link separating the other two phases. The modulus of elasticity of normal-weight aggregate is significantly greater than that of the cement paste, as shown in Figure 13.1. The heterogenous structure of concrete not only affects its behavior under low stresses, as discussed in Section 13.2, but, in fact, dominates the behavior of concrete at all levels of stress.

It is well established that much of the physical nature of the response of concrete in compression can be described in terms of microcracks that can be observed at relatively low magnifications. These microcracks form at the paste–aggregate boundary and through the mortar constituent of concrete as compressive strain increases. Internal cracks and flaws exist even before loads are applied. Some of these are due to segregation and bleeding, particularly beneath large pieces of aggregate or reinforcing steel. However, most of these microcracks are bond cracks at the interfacial transition zone, which are due not only to differences in elastic moduli between the hardened cement paste and the aggregate, but also to different coefficients of thermal expansion and different responses to changes in moisture content. The incompatibility in elastic moduli can lead to considerable stress concentrations under the differential volume changes caused by continued hydration, temperature differentials, or drying of the concrete. Since the strength of the interfacial transition zone is lower than the strength of the cement paste matrix, cracks tend to form in this region.

Studies have shown that there is a close correlation between the applied strain and the degree of microcracking within concrete, as illustrated in Figure 13.20. At zero load, bond microcracks exist due to the reasons just discussed. For stresses up to about 30% of the ultimate, little additional microcracking is observed (Figure 13.21). Bond microcracking begins to increase at stresses above 30 to 40% of the compressive strength. At about 70% of the compressive strength, cracks begin to form through the mortar constituent of the concrete, bridging bond cracks on individual aggregate particles. Bond and mortar microcracking continue at an increasing rate up to the peak stress. For strains exceeding the strain corresponding to the peak stress, some of the microcracks become visible to the unaided eye, while bond and mortar microcracks continue to form. As shown in Figure 13.22a, the density of bond and mortar microcracks observed in concrete becomes progressively greater with increasing compressive strain, especially for strains corresponding to the descending portion of the stress–strain curve.

Other studies indicate that cement paste undergoes microcracking that is strikingly similar to that in concrete, only at a much smaller scale. Within cement paste, the groundmass or undesignated product (UDP) (see Chapter 4) behaves as the matrix, and the harder phases, unhydrated cement particles and calcium hydroxide, behave as microaggregates. Prior to loading, microcracks are observed within the UDP and at the boundary between UDP and the harder phases. As compressive stress is applied, cracks form preferentially (but not exclusively) within the softer phases, as shown in Figure 13.23. The density of microcracks of the type shown in Figure 13.23 is much higher than the density of bond and mortar microcracks shown in Figure 13.20, as demonstrated in Figure 13.22b. Like the larger bond and mortar microcracks in

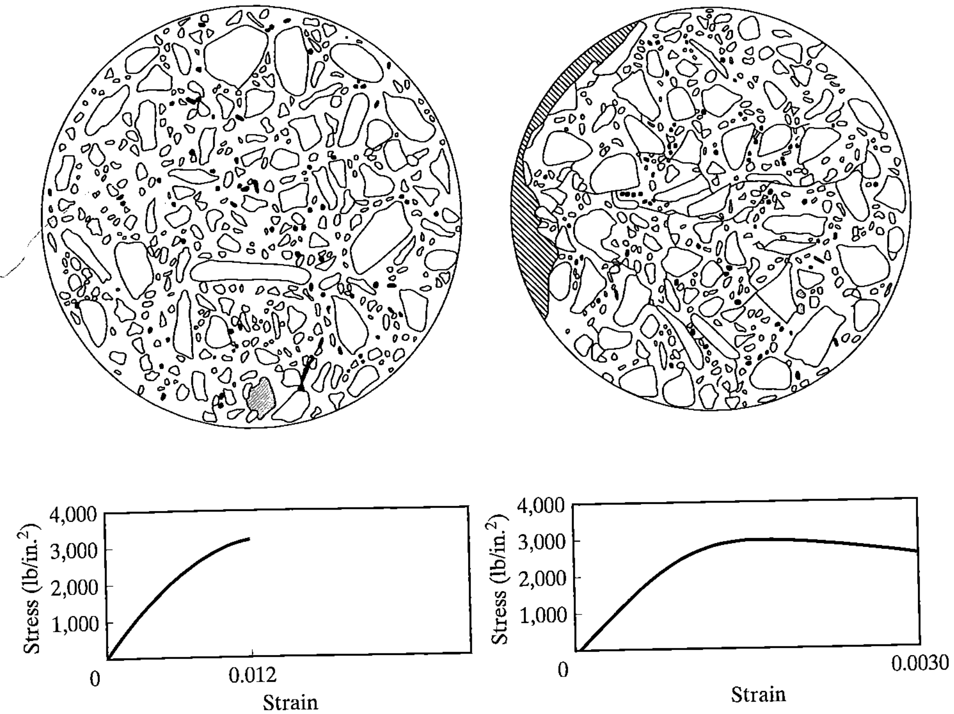


FIGURE 13–20

Cracking maps and stress–strain curves for concrete in uniaxial compression. [From S. P. Shah and F. O. Slate, in *The Structure of Concrete*, eds. A. E. Brooks and K. Newman, Cement and Concrete Association, London, pp. 82–92 (1968). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

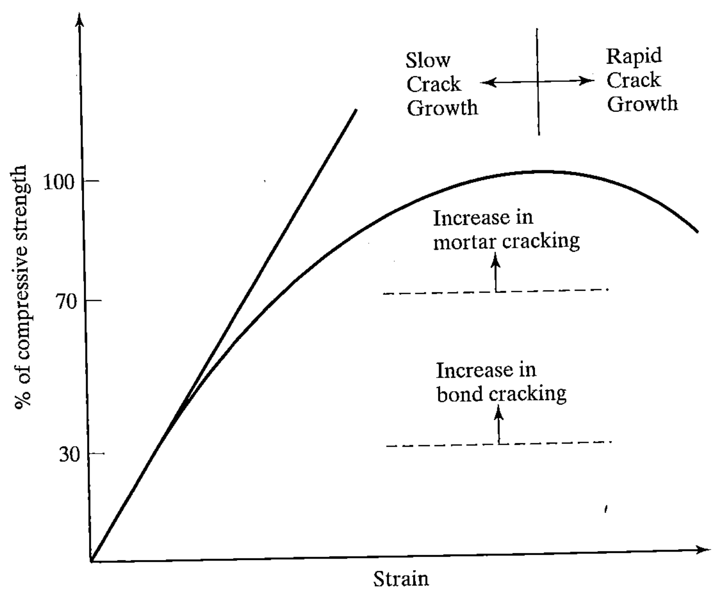


FIGURE 13.21

Diagrammatic stress–strain curve of concrete in compression. [From ACI Committee 224, *Journal of the American Concrete Institute*, Vol. 69, No. 12, pp. 717–753 (1972).]

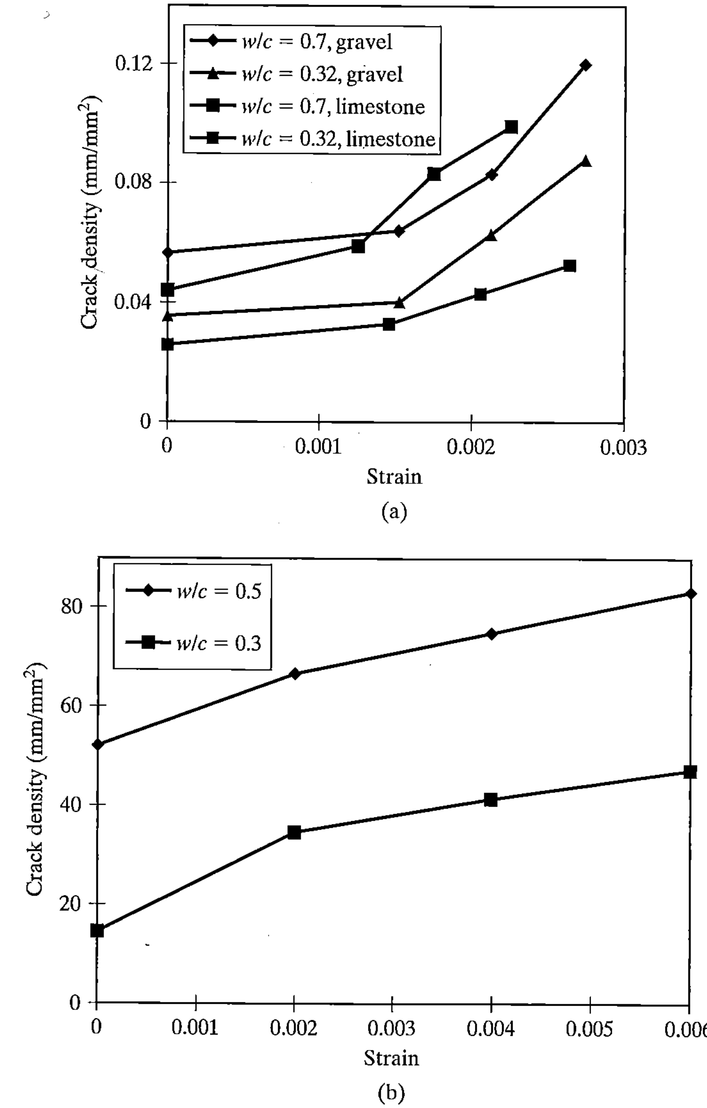


FIGURE 13.22

Relationship between microcracking and compressive strain for (a) concrete [data from R. L. Carrasquillo, F. O. Slate, and A. H. Nilson, *Journal of the American Concrete Institute*, Vol. 78, No. 3, pp. 179–186 (1981)] and (b) bulk cement paste [data from M. N. Abou-Zeid, *Microstructural Analysis of Cement Paste in Compression*, Ph.D. Thesis, University of Kansas, Lawrence (1994).]

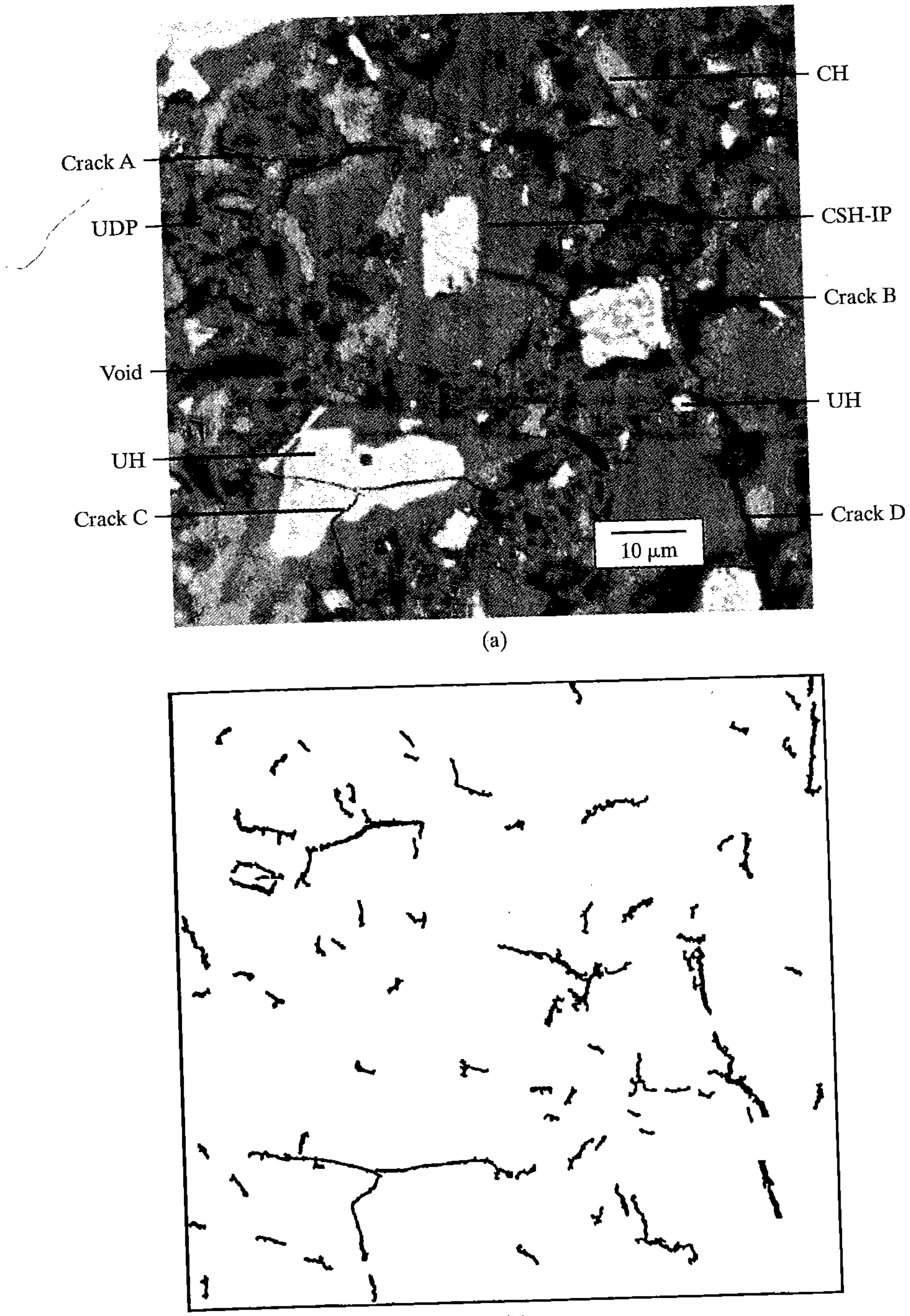


FIGURE 13.23

Microcracking in bulk cement paste $w/c = 0.5$ loaded to a compressive strain of 0.006: (a) backscattered electron image (UH = unhydrated cement, CSH-IP = inner product); (b) crack map. (From D. Darwin, M. N. Abou-Zeid, and K. Ketcham, *Cement and Concrete Research*, Vol. 25, No. 3, pp. 605–616 (1995).]

concrete, the density of microcracks in cement paste increases with applied compressive strain, although the rate of increase appears to be greater at low strains than at high strains.

Because of the highly heterogeneous nature of concrete, the localized stresses and strains are quite different from the nominal applied stresses and strains. This is shown in Figure 13.24, which is based on studies using photoelastic coatings on the surface of a concrete specimen in compression. The localized strains may be as much as 4.5 times the average strains, and localized stresses may be more than twice as high as the average stresses. The largest strains occur at the contact zone between paste and aggregate, accounting for the early formation of bond microcracks.

Figure 13.25 shows the idealized stresses around a single aggregate particle in concrete under compressive stress. For the normal case where the particle is stiffer than the matrix, stress analysis indicates the following order of failure: (1) tensile bond failure, (2) shear bond failure, (3) tensile matrix failure, and (4) occasional aggregate failure. This small-scale anisotrophy, brought on by the presence of aggregate, helps explain both the deformation and failure characteristics of concrete (and, if unhydrated cement particles and calcium hydroxide are treated as microaggregates, of cement paste). Thus, even in compression, there will be some regions around aggregate particles subjected to tensile stress. When the stress reaches the material tensile strength, a crack will start to grow. Compressive failure is thus preceded by progressive cracking, largely parallel to the axis of loading. (This is, of course, affected considerably by the end conditions of the particular compressive test, as discussed earlier and in Chapter 14.) Failure is eventually due primarily to the lateral tensile stresses produced by the microstructural failure of the material. For concrete, at the macroscopic level, failure is distributed over a fairly wide region, principally due to the large size of coarse aggregate particles compared to the specimen. For cement paste specimens, the macroscopic

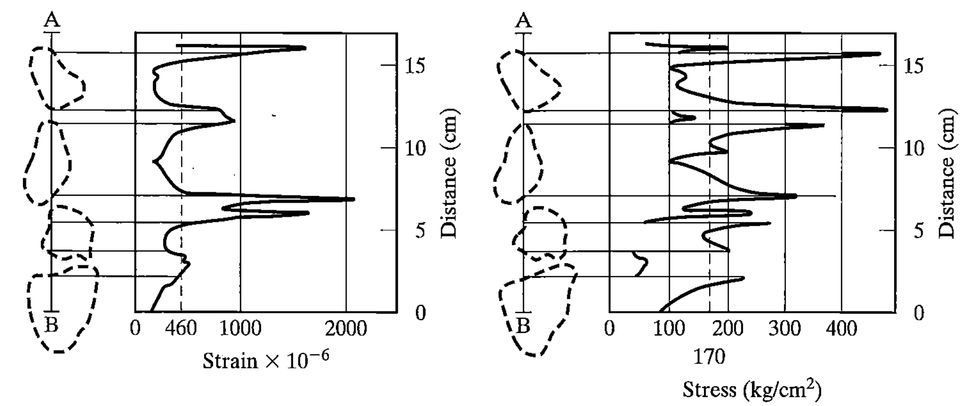


FIGURE 13.24

Distributions of strain and stress in the mortar and aggregate phases of a concrete specimen under uniaxial compression. Dashed lines represent the average values. Broken outlines represent aggregate particles. [From P. Dantu, *Annales de L'Institut Technique du Bâtiment et des Travaux Publics*, Vol. 11, No. 121, pp. 55–77 (1958).]

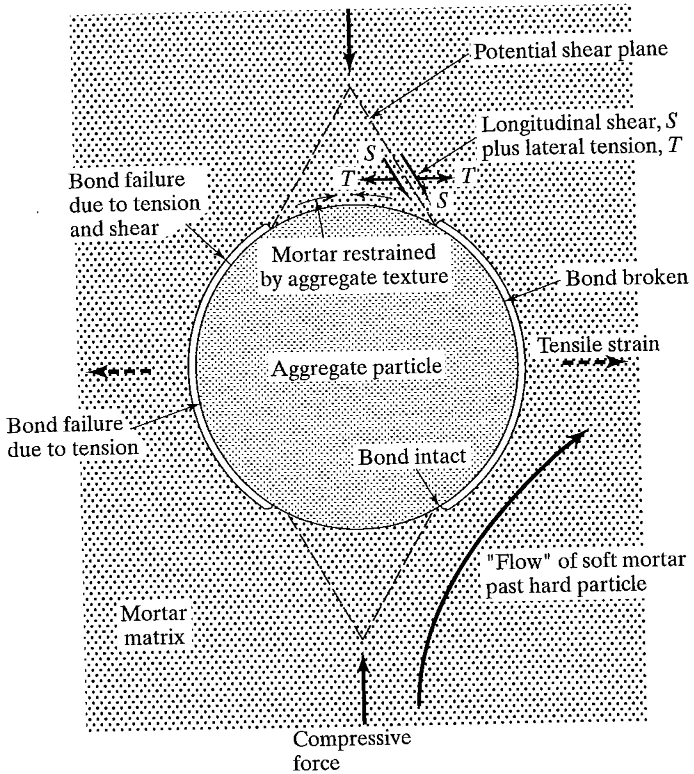


FIGURE 13.25

Idealization of stresses around a single aggregate particle. [From G. W. D. Vile, *The Structure of Concrete*, ed. A.E. Brooks and K. Newman, Cement and Concrete Association, London, pp. 275–288 (1968). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.)

failure region is generally much more localized and consists of a few longitudinal cracks.

The nature of the uniaxial compressive response of concrete depends not only on material properties, but on the direction of the loading with respect to the direction of casting. As shown in Figure 13.26, loading parallel to the direction of casting results in a more ductile response as the voids below the aggregate particles, which are caused by bleeding, are closed due to the application of the compressive stress. Under loading perpendicular to the direction of casting, the concrete possesses less ductility since the initial flaws are oriented parallel to the direction of loading and are opened by the tensile stresses that are induced within the concrete. As shown in Figure 13.26, compressive strength is affected only a few percent, indicating that the peak load is governed principally by local compressive stresses within the cement paste constituent along bearing surfaces in contact with the coarse aggregate particles. Large-scale tests on

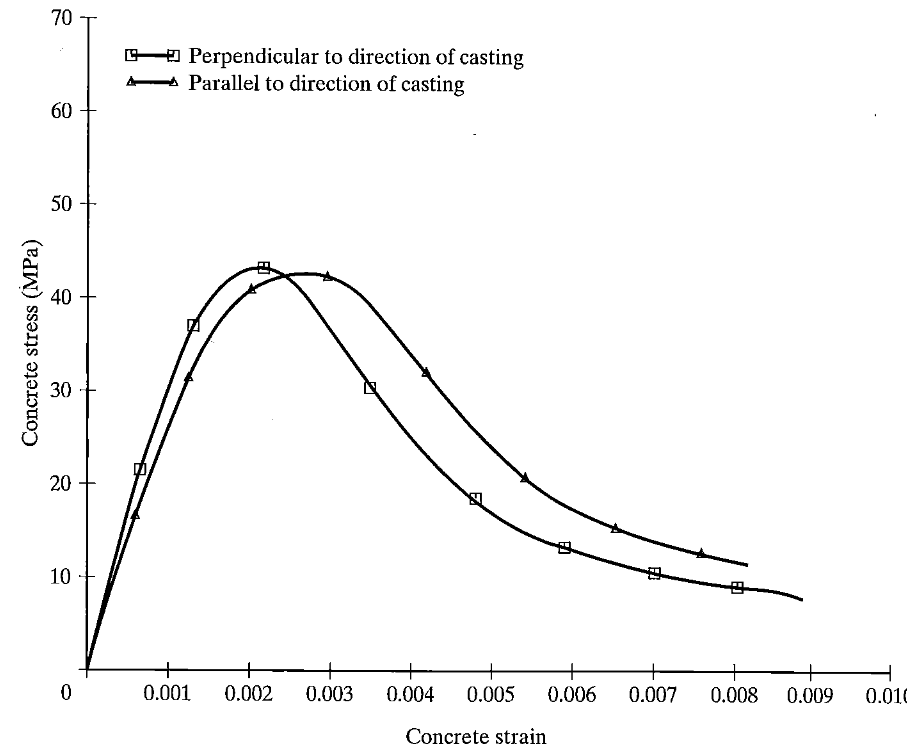


FIGURE 13.26

Influence of initial anisotrophy on the uniaxial stress–strain curve in compression. [Adapted from J. G. M. van Mier, *Strain-Softening of Concrete Under Multiaxial Loading Conditions*, Eindhoven University of Technology, Eindhoven, The Netherlands, (1984).]

cores, however, indicate that the compressive strength of concrete loaded perpendicular to the direction of casting ranges from 90 to 100% of the strength of concrete loaded parallel to the direction of casting, with an average between 92 and 96%.

The strength of the interfacial transition zone has a measurable effect on the compressive strength of concrete, although the effect is smaller than for the tensile properties of concrete. The role of the ITZ in compressive strength is illustrated in Figure 13.27. Figure 13.27a shows the results of tests in which the coarse aggregate particles were covered by a thin layer of hard plastic. The coating had little effect on the modulus of elasticity of the aggregate particles, but greatly reduced the bond strength at the paste–aggregate boundary. As illustrated, the large reduction in paste–aggregate strength reduced the compressive strength by only about 10%. In an analytical study, summarized in Figure 13.27b, the interfacial strength between a single aggregate particle and surrounding mortar was varied from perfect bond to zero. In the case of perfect bond, the compressive strength of the model increased by only 4% compared to the strength obtained with normal interfacial strength. For zero interfacial strength, the compressive strength decreased by just 11%. Overall, these results show that, while the

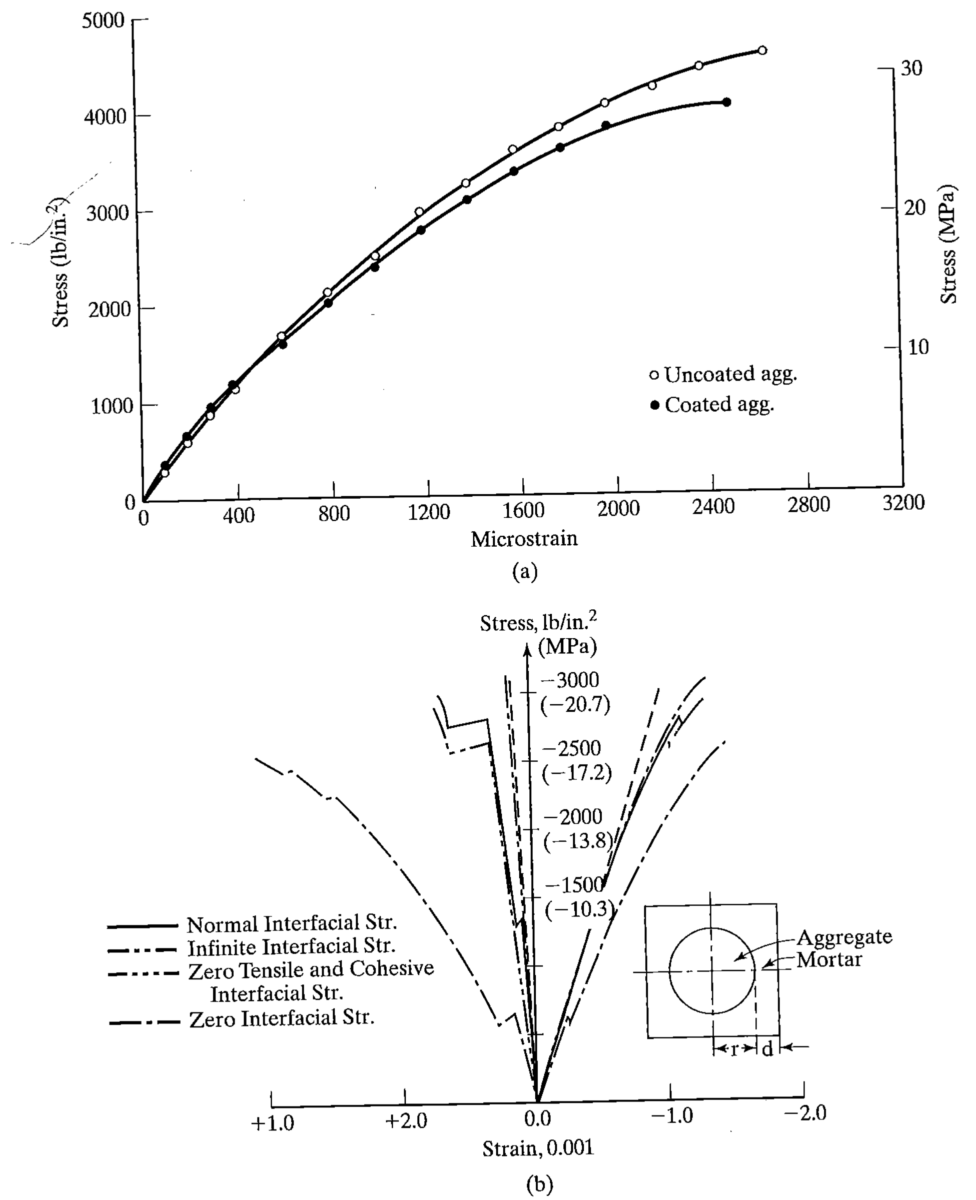


FIGURE 13.27

(a) Stress–strain curves for concrete comparing the effects of uncoated and coated coarse aggregate. [From D. Darwin and F. O. Slate, *Journal of Materials*, ASTM, Vol. 5, No. 1, pp. 86–98 (1970).] (b) Stress–strain curves for finite element models of concrete with different values of mortar–aggregate bond strength. [From A. Maher and D. Darwin, *Proceedings, First International Conference on Mathematical Modeling*, St. Louis, Vol. III, pp. 1705–1714 (1977).]

strength of the ITZ does affect the compressive response of concrete, it is overshadowed by the properties of the cement paste and aggregate constituents of concrete and the heterogeneous nature of the composite.

Stress–Strain Curves

Stress–strain curves for concretes of different strengths made with similar materials are shown in Figure 13.28. This figure shows that, as discussed in Section 13.2, the modulus of elasticity increases with increasing strength. The figure also shows that the strain corresponding to the peak compressive stress increases with increasing strength, while the descending portion of the stress–strain curve becomes progressively steeper.

It should be noted here that with a conventional, rather "soft" (low stiffness) testing machine, concrete in compression will appear to fail "instantaneously" when the maximum load is reached, making it impossible to record the descending branch of the stress–strain curve. The descending branch can be obtained only with functionally stiff testing machines that either (1) do not store large amounts of energy that cause the loading frame to rebound after the specimen reaches its peak stress or (2) respond by reducing the load based on feedback from the test specimen (closed-loop testing) that is used to impose a specified strain, rather than stress, history.

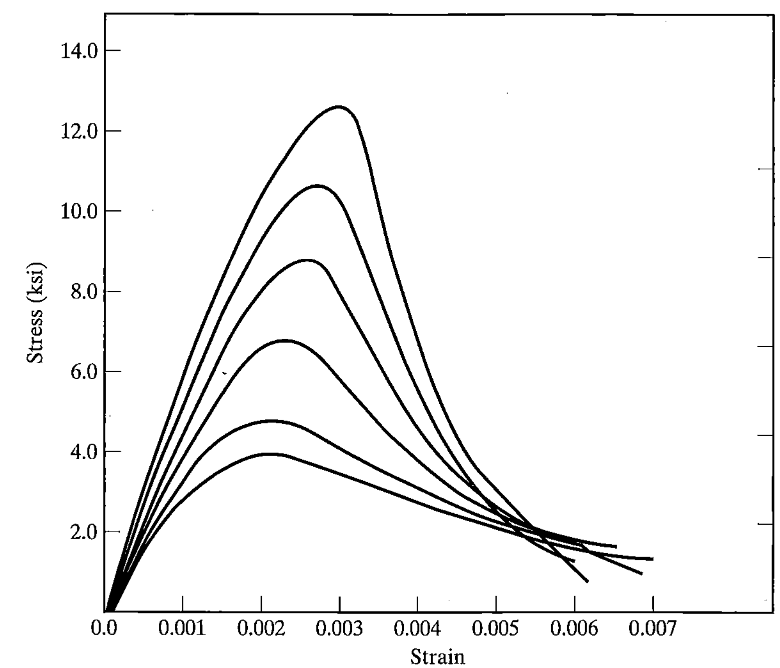


FIGURE 13.28

Stress–strain curves for concretes with different strengths. [From S. H. Ahmad and S. P. Shah, *Journal of the American Concrete Institute*, Vol. 82, No. 5, pp. 634–647 (1985).]

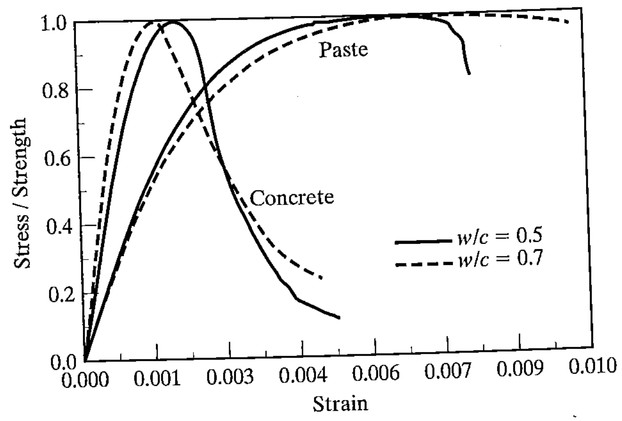


FIGURE 13.29

Normalized stress–strain curves for concretes and bulk cement pastes with the same w/c ratios. [Adapted from J. L. Martin, D. Darwin, and R. E. Terry, *SM Report*, No. 31, University of Kansas Center for Research (1991).]

It is instructive to compare the shapes of stress–strain curves for concretes and cement pastes with the same water–cement ratios. Figure 13.29 compares stress–strain curves for cement pastes and concretes with w/c ratios of 0.5 and 0.7; in this case, stress is normalized with respect to strength. The curves show that cement paste exhibits significantly more ductility and has a broader high stress region than does concrete. However, unlike the concrete, the cement paste with the lower w/c ratio and higher strength exhibits a reduction in the strain corresponding to the peak stress compared to the weaker material. The differences in the behavior between concrete and its paste constituent are due to factors discussed previously in this section, including the high stress concentrations in the vicinity of the aggregate particles, which can subject a portion of the cement paste to significantly higher stresses than the nominal stress applied to the concrete, and the greater heterogeneity of concrete, which results in increased transverse tensile stresses.

The formation of microcracks has a significant effect on the lateral and volumetric strain in concrete subjected to compression. The beginning of mortar microcracking corresponds to an apparent increase in Poisson's ratio, as shown in Figure 13.30 by the rapid increase in the ratio of ε_3 to ε_1 above the stress σ_A. At the onset of mortar microcracking, the gross volume of concrete in compression also begins to increase. Clearly, these effects are due to the great increase in cracking that is taking place. Again, this behavior is closely related to the composite nature of concrete, since for hardened cement paste alone in compression, the volume decreases continuously until failure occurs.

13.5 CYCLIC LOADING

When subjected to cyclic loading in either tension or compression, concrete accumulates additional strain and deflection with each cycle. Figure 13.31a shows portions of stress–strain curves for concrete in compression when subjected to a peak stress equal to 67.5% of the compressive strength. Figure 13.31b shows the load versus crack mouth opening displacement[3] (CMOD) behavior of a fracture specimen subjected to cyclic

[3]The CMOD is equal to the total separation of opposite sides of a crack where the crack intersects the exterior of a member, at the "crack mouth."

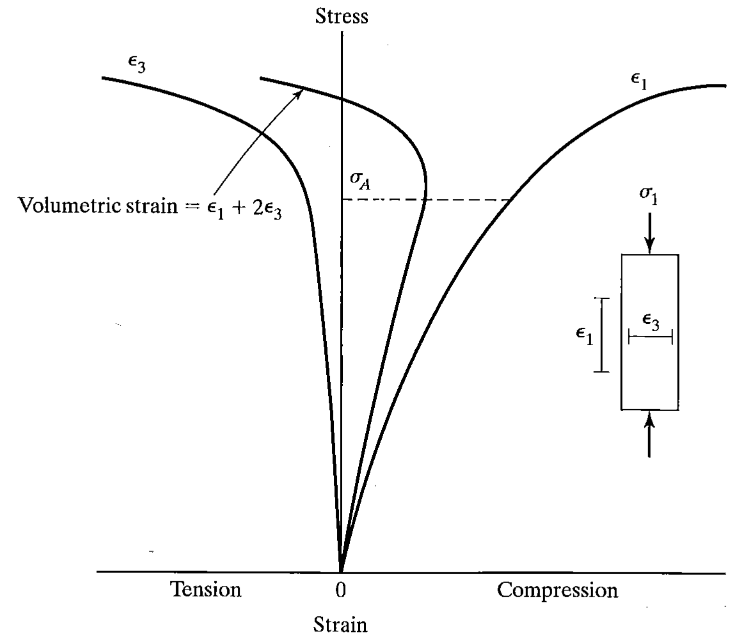


FIGURE 13.30

Stress–strain curves for typical concrete loaded in uniaxial compression. [Adapted from K. Newman and J. B. Newman, *Structure, Solid Mechanics, and Engineering Design*, ed. M. Te'eni, John Wiley and Sons, Limited, pp. 963–995 (1971).]

loading in which the peak load on every fifth cycle is increased compared to the background loading.

During cyclic loading in compression, the shape of the stress–strain curve changes, from concave downward initially to concave upward, as shown in Figure 13.31a. This change is accompanied by a reduction in the secant modulus of elasticity (based on the maximum repeated stresses and strains), as shown in Figure 13.32a. The accumulation of strain is rapid during the initial cycles, slows for the intermediate cycles, and then increases again for the final cycles just prior to failure, as shown in Figure 13.32b. The shape of the stress–strain curves with increasing cycles of compression indicates that internal surfaces, such as across microcracks, are in contact during the initial cycles, but become progressively more separated upon unloading in subsequent cycles. As the load increases during each subsequent cycle, the concrete becomes stiffer (and the stress–strain curve becomes steeper) as more and more of these surfaces come in contact. The strain accumulated under cyclic compression consistently exceeds the value of strain obtained for similar specimens subjected to sustained loading equal to the mean stress of the cycles. In fact, even when the nonlinear relationship between stress and creep strain is accounted for (for specimens subjected to high stresses), the strain for specimens under cyclic loading exceeds the expected total strain under the (higher than mean) equivalent sustained stress. This indicates that cyclic loading itself

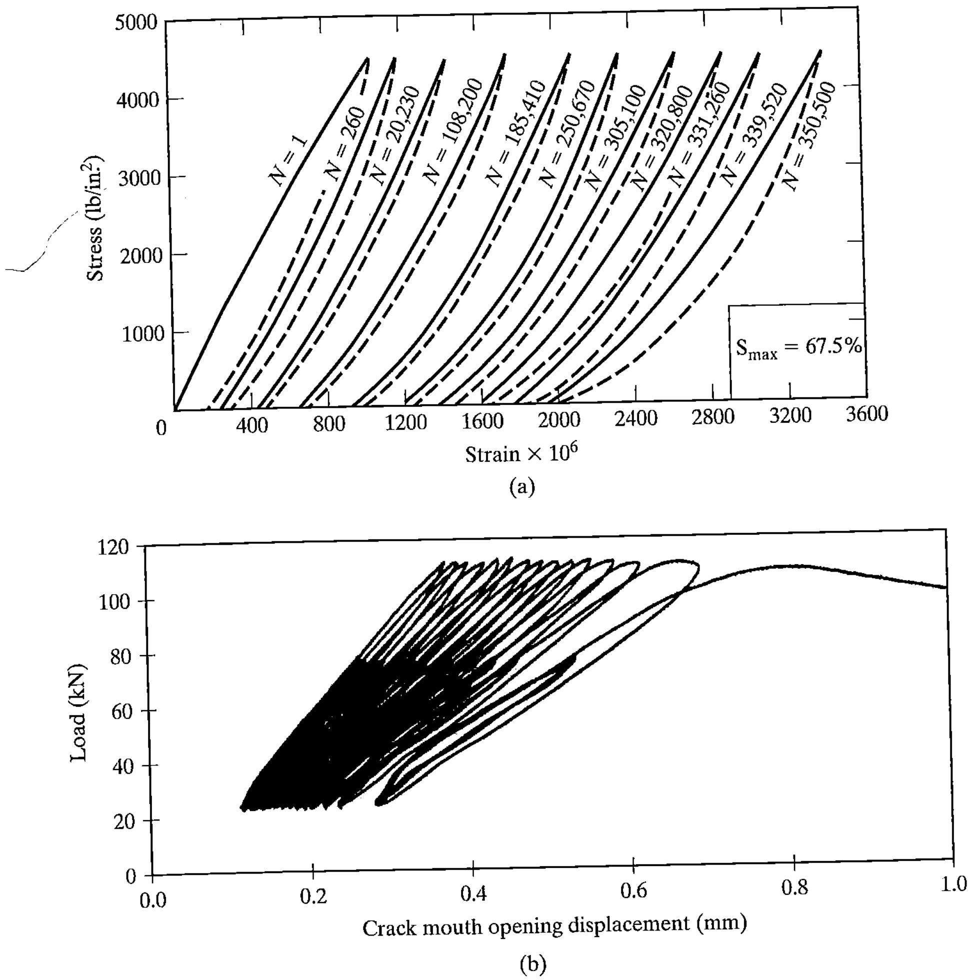


FIGURE 13.31

(a) Variation of compressive stress–strain curve with number of cycles. [From E. W. Bennett and M. K. Razu, *Structure, Solid Mechanics, and Engineering Design*, Part 2, ed. M. Te'eni, John Wiley and Sons, Limited, pp. 1089–1102 (1971).] (b) Load verses crack mouth opening displacement for fracture specimen under cycles of load. [From V. Slowik, G. A. Plizzari, and V. E. Saouma, *ACI Materials Journal*, Vol. 93, No. 3, pp. 272–283 (1996).]

causes structural changes in concrete that may consist of both damage and viscous flow. For short-term tests (4 hours or less), the extra strain can be measured for cyclic loads as low as 40% of the static strength.

The load–CMOD curves shown in Figure 13.31b are generally similar to load-deflection curves obtained under cyclic compression; however, the shape of the curves remains concave downward, indicating that upon unloading the crack surfaces remain in contact as the crack is closed.

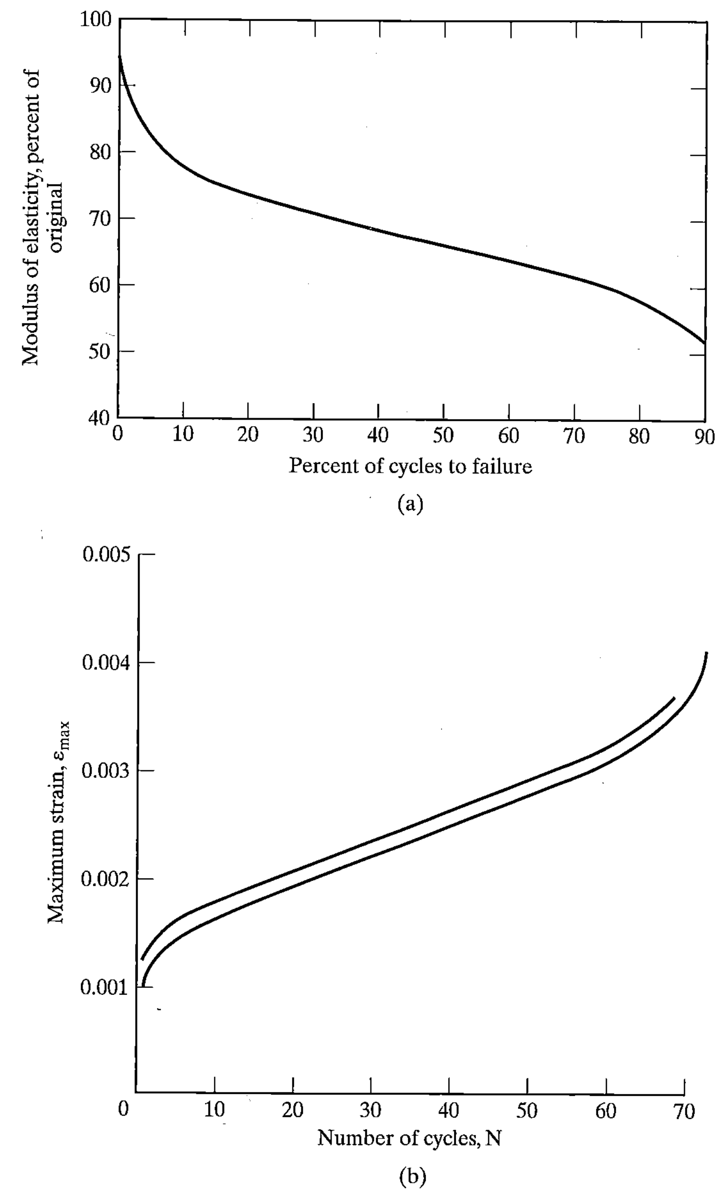


FIGURE 13.32

(a) Reduction in secant modulus of elasticity as the result of repeated loads. [From D. A. Linger and H. A. Gillespie, "A Study of the Mechanism of Concrete Fatigue and Fracture," *Highway Research News*, No. 22, pp. 40–51, (February 1966). Highway Research Board, National Research Council, Washington, D. C.] (b) Increase in peak strain for mortar in compression as the result of repeated loads. [Adapted from A. Maher and D. Darwin, *SM Report* No. 5, University of Kansas Center for Research (1980).]

The fundamental mechanisms responsible for failure under cyclic loading may be explained using the concepts of stress concentration and fracture mechanics developed earlier in this chapter. Concrete contains a variety of internal flaws and cracks that form during the hydration process and become more extensive during periods when drying shrinkage occurs. These cracks are often found at the ITZ. When concrete is loaded in either tension or compression, the stresses at the tips of these cracks can become very large, exceeding the cohesive strength of the cement paste or the ITZ, even though the nominal stress in the concrete is quite low. Under repeated cycles of loading, these cracks will tend to grow. This is demonstrated for the two cyclic loading regimes in Figure 13.31 by the formation of hysteresis loops. For load–deflection curves, the area within each loop is equivalent to the irreversible energy of deformation, some of which represents sliding across surfaces and some of which represents the energy used to extend cracks. As shown in Figure 13.31, the area within the hysteresis loops decreases at first with successive load cycles, but then begins to increase prior to failure. At the beginning, crack growth is slow, and there may also be some viscous flow. The growth of microcracks interacting with aggregate tends to stabilize the larger cracks that form. However, the energy supplied by the repeated loadings eventually increases the damage at the crack tips, and the cracks continue to propagate until failure occurs.

Fatigue

Fatigue is the phenomenon by which a material is caused to fail by the repeated application of loads that are not large enough to cause failure in a single application. This implies, as just discussed, that under repeated stresses, some internal progressive permanent structural change takes place in the concrete. This change, which may be referred to as fatigue damage, not only involves the growth of microcracks, but also viscous flow (or creep). It has been found that fatigue occurs under compressive, tensile, and flexural loading. As with metals, the factors that affect the static strength of concrete have a similar effect on fatigue.

Cyclic loading of concrete is usually separated into concrete behavior under *low cycle fatigue* and *high cycle fatigue*. The distinction is more practical than real, since low cycle fatigue occurs with cycles of relatively high stresses, whereas high cycle fatigue occurs under lower stresses. Low cycle fatigue usually refers to the type of loading that might occur under major excursions of loading, such as may occur in an earthquake, whereas high cycle fatigue usually represents material behavior under cycles that occur throughout the life of a structure. ACI Committee 215 defines high cycle fatigue as loading that leads to failure in 100 cycles or more.

Fatigue Strength Fatigue data are usually presented graphically by an S–N diagram. This is a plot of repeated stress, S (expressed as the stress ratio, or percentage of the static strength), versus the logarithm of the number of cycles to cause failure, N. Unlike most metals, concrete does not appear to have a *fatigue limit* (i.e., the S–N curve does not become horizontal), at least out to 10^7 cycles of loading. The fatigue strength is the applied stress required to cause failure after a given number of cycles. At 10^7 cycles, it is approximately 55% of the static strength, regardless of whether the loading is in

compression, tension, or flexure. However, a single S–N curve is inadequate to describe fatigue behavior; it is also necessary to take into account the large amount of scatter inherent in fatigue tests of concrete. Therefore, it is much more useful to present fatigue data in terms of the probability of failure, p, as shown in Figure 13.33. The usual S–N curve is that shown for $p = 0.5$.

At stresses below about 75% of the static strength, not only does the fatigue strength appear to be independent of the type of loading, it also seems to be independent of the details of the mix design. Cement, mortar, and concrete seem to have approximately the same S–N diagrams, which do not change significantly for different ages, w/c ratios, aggregates, air entrainment, etc. At stresses above about 75% of the static strength (representing low cycle fatigue), cement paste appears to have a longer fatigue life than does mortar or concrete with the same w/c ratio.

In contrast to the lack of sensitivity to the details of mix design, concrete is very sensitive to the *range* of the cyclic stress (i.e., the difference between the highest and lowest stresses). It has been found that as the stress range decreases, the maximum stress that can be sustained for a given fatigue life increases. This is illustrated in Figure 13.34, which is recommended for use in design for plain concrete in both tension and compression by ACI Committee 215. This figure allows the determination of the maximum stress the concrete can withstand for 10^6 load cycles for a given minimum stress; for zero minimum stress, the conservative estimate is that the maximum stress (and total stress range) for 10^6 cycles is 50% of the static strength, f_c'. As the minimum stress increases (reducing the stress range), the maximum stress increases. In the example shown in the figure, the minimum stress is increased from 0 to $0.15 f_c'$, producing a stress range of $0.425 f_c'$ and a maximum stress of $0.575 f_c'$ for 10^6 cycles.

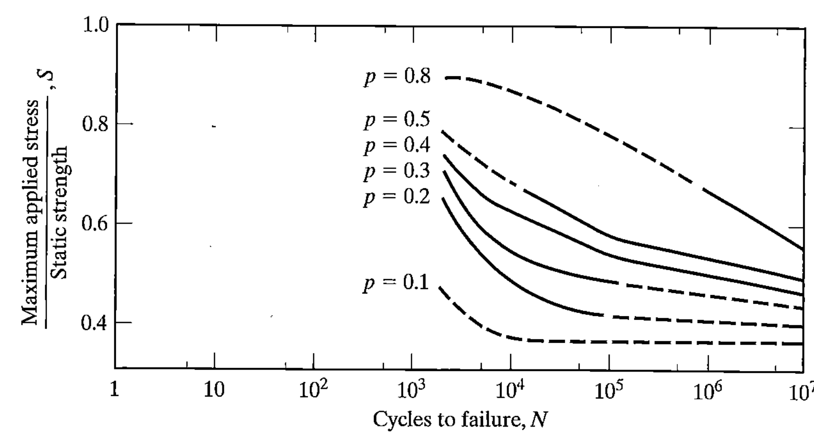


FIGURE 13.33

Typical set of fatigue curves for probabilities of failure for plain concrete subjected to reversed flexural loading. [From J. T. McCall, *Journal of the American Concrete Institute*, Vol. 55, No. 2, pp. 233–245 (1958).]

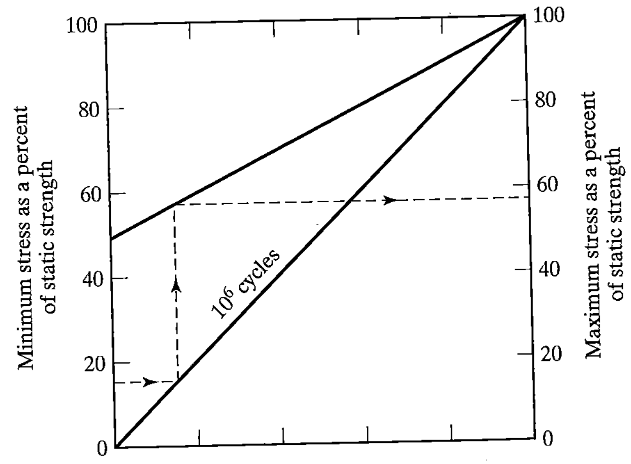


FIGURE 13.34

Fatigue strength of plain concrete in tension, compression, or flexure. [From ACI Committee 215, "Considerations for Design of Concrete Structures Subjected to Fatigue Loading (ACI 215R-74) (Revised 1992)," *ACI Manual of Concrete Practice*, Part 1, American Concrete Institute, Farmington Hills, MI (2001).]

The frequency of cyclic loading appears to have little effect on fatigue strength, as long as the maximum stress level is less than about 75% of the static strength. At higher loadings, an increased strain rate results in higher fatigue strengths, while the effects of creep reduce the strength for low rates of cyclic loading.

Loading History Loading history has an effect on fatigue strength under both high and low stresses. For high cycle fatigue, it is convenient to use the *Palmgren–Miner rule* as an approximation for concrete. This suggests that failure will occur when

$$\sum_i \frac{n_i}{N_i} = 1 \tag{13.20}$$

where n_i is the number of cycles of load at some stress condition and N_i is the number of cycles required to cause failure at that condition. That is, the rule assumes that there will be a linear accumulation of damage due to each loading cycle, with the accumulation depending on the severity of the cycle. Application of the Palmgren–Miner rule is illustrated in Figure 13.35. For load conditions 1 and 2, with mean stress σ_{m1} and σ_{m2}, respectively, the sum of the *life fractions* leading to failure is n_1/N_1 plus n_2/N_2. When the sum of life fractions satisfies Eq. (13.20), failure will occur. This hypothesis is not always conservative, and it must be applied with considerable caution.

The Palmgren–Miner rule is unable to account for certain effects of load history. For instance, it has been found that frequent rest periods during a fatigue test may increase the fatigue strength, although the improvement increases with the length of rest only up to about five min. This is shown in Figure 13.36, obtained from tests in which rest periods of varying length were observed after every 4,500 load cycles. During these rest periods, the load was maintained at the lower limit of the repeated load. This effect is probably due to some stress relaxation during the rest period. It would appear that a brief period at a high cyclic stress level, followed by a lower stress level to failure results in a greater fatigue life than obtained for concrete subjected only to the lower stress continuously.

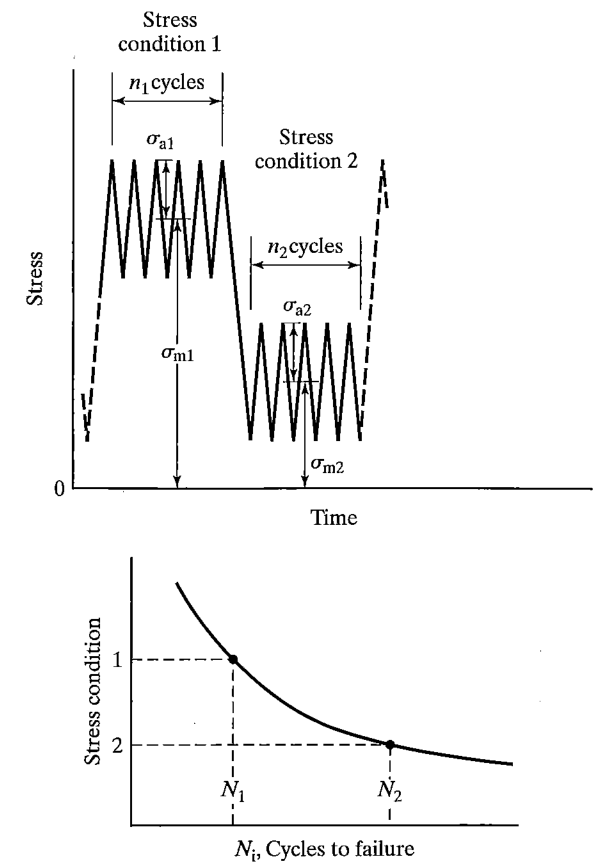


FIGURE 13.35

Application of the Palmgren–Miner rule to predict fatigue failure for a material subjected to varying load conditions.

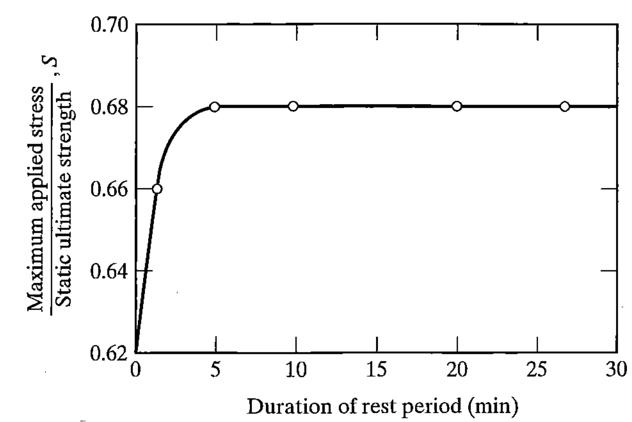


FIGURE 13.36

Effect of rest period on fatigue strength at 10 million cycles. [From H. K. Hilsdorf and C. E. Kesler, *Journal of the American Concrete Institute*, Vol. 63, No. 10, pp. 1059–1076 (1966).]

Also, monotonic or cyclic loading below the fatigue limit (i.e., below the load that would cause failure after 10^7 cycles) tends to improve both the fatigue and static strength by 5 to 15%, possibly due to some densification of the structure. Studies indicate that the maximum cyclic load to achieve this benefit is 40% of the static strength. Regular spikes of high load, such as those used for the fracture specimen in Figure 13.31b, result in an acceleration of the rate of damage and a reduction in the fatigue life. In the case illustrated in Figure 13.31b, the compliance of the specimen increased much more rapidly following application of the high load cycle.

It has also been found that stress gradients have a significant effect on fatigue life. The fatigue strength of nonuniformly stressed specimens (simulating beams) is about 15% higher than that obtained for uniformly loaded specimens. A similar effect is found for static loading, where it has been observed that eccentric loading retards the growth of microcracks. Finally, it has also been shown that fatigue strength is improved by the application of lateral confining pressure.

13.6 MULTIAXIAL STATES OF STRESS

So far, the discussion has emphasized the response of concrete to uniaxial states of tension or compression. In practice, however, concrete is rarely subjected to uniaxial stress, and in the general case, concrete might be subjected to a combination of compressive, tensile, and shearing stresses, all acting simultaneously. The behavior of concrete under these combined stresses is, unfortunately, still not completely understood. As a result, a generally accepted criterion for concrete failure has not been developed, although a number of both empirical and theoretical failure laws have been proposed. The vast majority of the studies on the behavior of plain concrete under multiaxial states of stress have involved tests in which the principal stress axes do not rotate.

Biaxial Loading

For biaxial loading, the most useful tests involve those performed on concrete plates or prisms in which load is applied on perpendicular surfaces using brush bearing platens (discussed earlier in the chapter), which limit the lateral restraint that would be caused by friction between the platen and the test specimen. The classic study performed by Kupfer, Hilsdorf, and Rusch remains one of the best available. Stress–strain curves for concrete under biaxial compression, tension–compression, and biaxial tension are shown in Figures 13.37a, b, and c, respectively. Overall strength under conditions of biaxial stress is shown in Figure 13.38 for concretes with strengths ranging from 18.6 to 57.6 MPa (2700 to 8350 psi).

As shown in Figure 13.37a, the application of biaxial compressive stress results in an increase in the strain corresponding to the peak stress. Strength is highest for a biaxial strength ratio of approximately 0.5 (Figure 13.38), decreasing as the biaxial compressive stresses approach equality. Under biaxial tension–compression, strength decreases as do the strains corresponding to the peak compressive and tensile stresses as the ratio of the tensile stress (σ_2) to compressive stress (σ_1) increases, as shown in Figure 13.37b. Under biaxial tension (Figure 13.37c), the tensile strength of concrete is unaffected by the multiaxial state of stress (see also Figure 13.38).

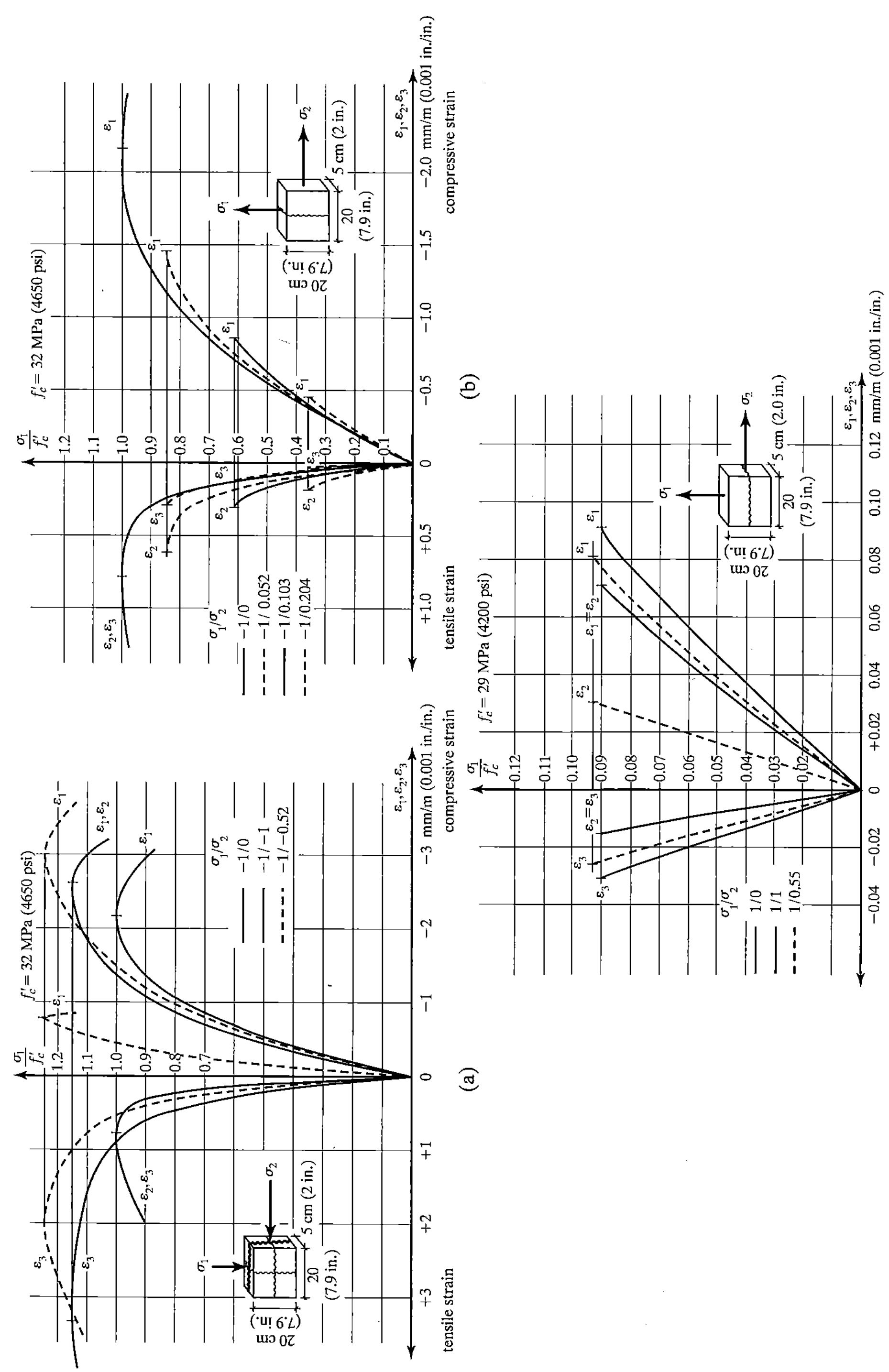


FIGURE 13.37
Stress–strain relationships for concrete under (a) biaxial compression, (b) combined tension and compression, and (c) biaxial tension. [Adapted from H. Kupfer, H. K. Hilsdorf, and H. Rusch, *Journal of the American Concrete Institute*, Vol. 66, No. 8, pp. 656–666 (1969).]

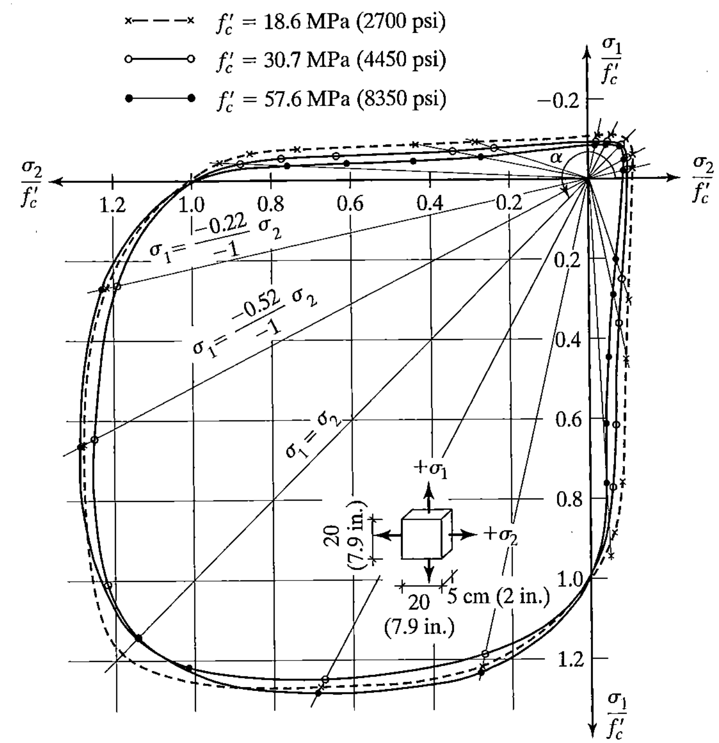


FIGURE 13.38

Biaxial strength of concrete. [Adapted from H. Kupfer, H. K. Hilsdorf, and H. Rusch, *Journal of the American Concrete Institute*, Vol. 66, No. 8, pp. 656–666 (1969).]

Failure modes under biaxial states of stress depend on the microscopic response of concrete to stress, as discussed earlier in the chapter. Under biaxial compression, the principal failure mode is tensile, with cracking parallel to the plane in which the biaxial stresses are applied. Under tension–compression, failure involves the formation of cracks perpendicular to the biaxial plane and perpendicular to the direction of the principal tensile stress. Under conditions of biaxial tension, cracks form perpendicular to the principal tensile stress, with no orientation of preference when the biaxial tensile stresses are equal.

Triaxial Loading

Concrete confined by reinforcement will be under triaxial restraint. Triaxial tests of concrete have involved, for the most part, tests of cylinders in which a lateral confining pressure is applied, with the principal compressive stress along the axis of the cylinder. One version of this test is described in ASTM C 801. Some triaxial tests have been performed with cubes loaded with either brush-bearing platens or fluid-loaded membranes.

Typical stress–strain curves for triaxial compression tests in which longitudinal stress is applied as the confining stress is held constant are shown in Figure 13.39. The compressive strength and the strain corresponding to the peak compressive stress increase as the confining stress is increased. Unlike uniaxial tests, in which the stress drops off rapidly after the peak compressive stress is attained, under triaxial compression,

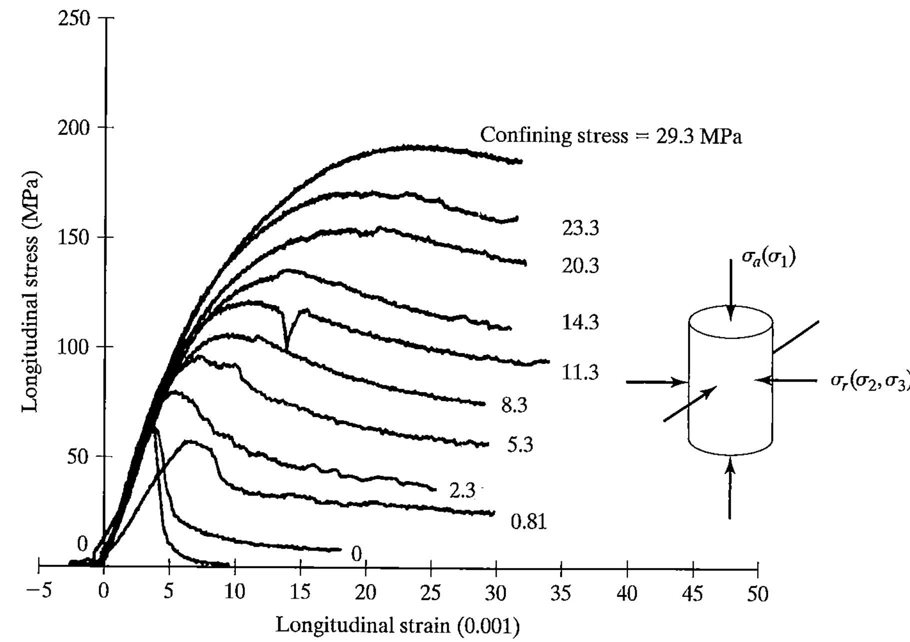


FIGURE 13.39

Longitudinal stress versus longitudinal strain for concrete under triaxial compression. [Adapted from J. Xie, A. E. Elwi, and J. G. MacGregor, *ACI Materials Journal*, Vol. 92, No. 2, pp. 135–145 (1995).]

the descending portion of the stress–strain curve tends to reach a nearly constant stress, referred to as the *residual strength*, which also increases with the confining stress. For confining stresses up to about 15% of the uniaxial compressive strength, the principal failure mode is a longitudinal splitting mode, as obtained in a uniaxial compression test. For higher confining stresses, damage is highly distributed, with little localized failure in evidence. Typical triaxial failure and residual strength envelopes are shown in Figure 13.40.

Under both biaxial and triaxial compression, for a given *w/c* ratio, strength increases more under the effects of the multiaxial compression as the aggregate content increases. In both tension and compression, the strength of concrete is affected much more by multiaxial states of stress than is cement paste. Analytical studies have demonstrated that even when the matrix (cement paste or mortar) is modeled as having no sensitivity to biaxial states of stress, the strength of the composite is affected, in the manner indicated in Figures 13.38 and 13.40, when aggregate inclusions are added. The various experimental investigations that have been carried out under multiaxial states of stress imply that compressive failure in concrete is governed by the strength of the paste, while tensile failure seems to be governed largely by the strength of the interfacial transition zone. A great deal remains to be learned about the behavior of concrete under multiaxial states of stress, including the importance of the stress history, rate of loading, and order in which the stresses are applied.

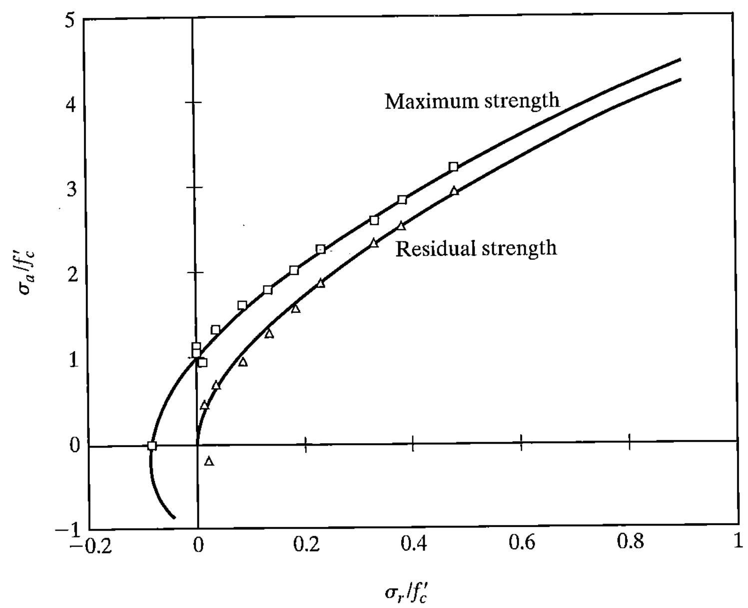


FIGURE 13.40

Maximum and residual strength surfaces for concrete in triaxial compression (σ_a = axial stress; σ_r = radial stress). [Adapted from J. Xie, A. E. Elwi, and J. G. MacGregor, *ACI Materials Journal*, Vol. 92, No. 2, pp. 135–145 (1995).]

13.7 FACTORS AFFECTING STRENGTH

Water/Cement Ratio

Even though the strength of concrete is dependent largely on the capillary porosity or gel/space ratio of the paste, these are not easy quantities to measure or predict. They are, therefore, not suitable for use in the mix design procedures outlined in Chapter 10. Fortunately, however, the capillary porosity of a properly compacted concrete at any degree of hydration is determined by the water/cement ratio. The paramount influence of the *w/c* ratio on the porosity of hardened cement paste at a given degree of hydration is shown in Figures 4.13b (Section 4.4) and 18.2 (Section 18.1). Therefore, in practice, the strength of a properly compacted concrete at a given age is controlled by specifying the *w/c* ratio. Even though Abram's *w/c* ratio law (Chapter 10) is not really a "law," in that it does not consider the degree of hydration, the air content of the concrete, or the effects of the aggregate, it is sufficiently correct in the usual range of *w/c* ratios encountered in practice to be a very useful tool. While most discussions involving *w/c* ratio involve compressive stress, tensile strength also increases with a decrease in the *w/c* ratio.

The *w/c* ratio law, as usually stated, is a simplification of the true state of affairs. One example of the limitation of the *w/c* ratio law is illustrated in Figure 13.41. Concrete that is not properly compacted will contain large voids, which contribute to its

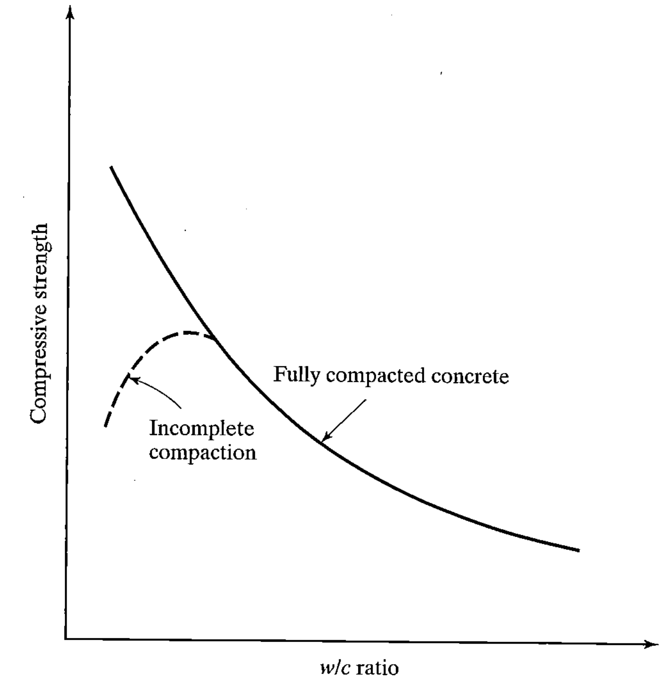


FIGURE 13.41

Relationship between compressive strength and *w/c* ratio.

porosity. Thus, at lower *w/c* ratios, where full compaction is difficult to achieve, Abram's law ceases to be valid. The point at which this occurs depends on the methods of compaction that are employed. In this regard, it might be pointed out that some sophisticated methods of compaction (used primarily in precasting operations) and the use of superplasticizers make possible the production of high-strength concretes using ordinary cement contents. That is, if full compaction can be attained, Abram's law is obeyed even at very low *w/c* ratios.

A better description of the role of the *w/c* ratio in determining strength was given by Gilkey:

> For a given cement and acceptable aggregates, the strength that may be developed by a workable, properly placed mixture of cement, aggregate, and water (under the same mixing, curing, and testing conditions) is influenced by: (a) ratio of cement to mixing water, (b) ratio of cement to aggregate, (c) grading, surface texture, shape, strength, and stiffness of aggregate particles, (d) maximum size of the aggregate.

Thus, in fact, the *w/c* ratio "law" is really a family of relationships for different mixtures, as shown in Figure 13.42, which emphasizes the effect of increasing heterogeneity as aggregate is added to cement paste in progressively larger quantities. Although there is general agreement that the *w/c* ratio law applies uniformly to ordinary, workable structural concrete [using graded aggregates up to a maximum size of 38 mm ($1\frac{1}{2}$ in.)], this law may not necessarily apply to mixes in which the maximum aggregate size or aggregate amount is very different from "ordinary" concrete, as in neat pastes or mortars on

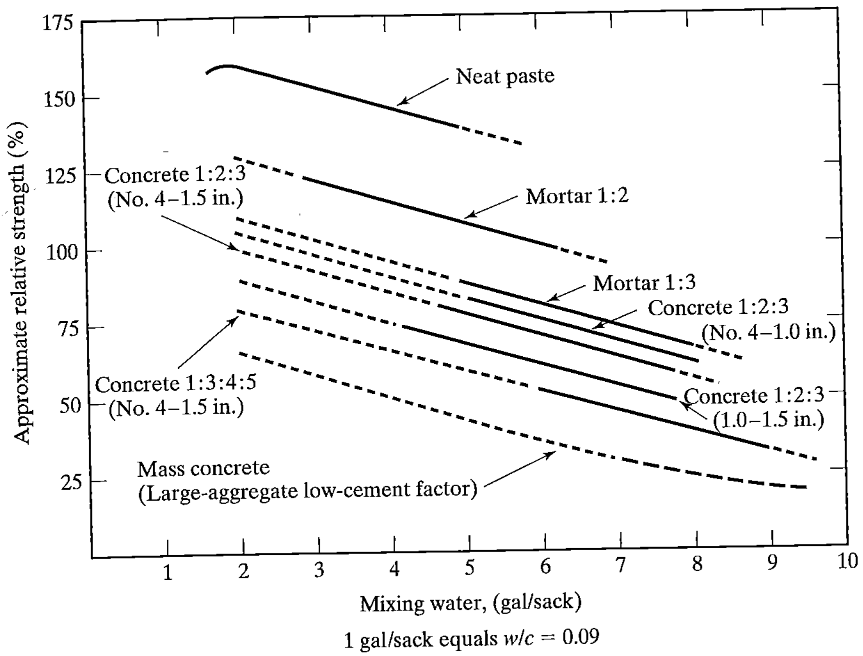


FIGURE 13.42

Water/cement ratio curves; approximate extreme workable ranges for a few mixtures of different proportions and/or coarse aggregate gradings. [From H. J. Gilkey, *Journal of the American Concrete Institute*, Vol. 57, No. 10, pp. 1287–1312 (1961).]

the one hand and mass concrete mixes on the other. The design of "unusual" concrete mixes based on a simple reliance on the *w/c* ratio dependency may be in serious error.

It is worth noting that while the *w/c* ratio, a measure of total porosity, is the principal controller of the strength of the cement paste constituent of concrete for given mixing and curing conditions, there are indications that the details of the porosity, as represented by the void size distribution, can have a significant effect on concrete strength. Reduction in the *w/c* ratio and the addition of silica fume not only results in a reduction in the total porosity, but also reduces the size of the maximum measurable voids within the hardened cement paste. Based on the discussion in Section 13.3, it should be clear that, if the voids (which act as flaws within the structure of cement paste) are reduced in size, the stress required to initiate cracking will be increased.

Although the *w/c* ratio is typically specified in mix designs, there is often a great deal of uncertainty over what the true *w/c* ratio is when the concrete is placed. In normal practice, only the slump test is used to indicate whether there is additional water in the mix, either from the aggregates or because of a deliberate addition of water to make the concrete easier to handle. It has thus been suggested that concrete be proportioned on the basis of performance standards, with no specification at all for the *w/c* ratio. In practice, this would mean specifying concrete on the basis of strength, since durability tests are too expensive and time consuming to be used routinely. Unfortunately, this

would be a backward step, since changes in strength may very well not correspond with changes in durability. In addition, the use of finer cements, certain admixtures, and special curing techniques could provide 7-day or 28-day compressive strengths that do not in any way represent the true quality of the concrete. Until some more reliable methods of specifying concrete performance in the field are developed, the *w/c* ratio remains the best tool for assuring satisfactory performance of concrete, in terms of both strength and durability.

Age

The knowledge that strength is controlled by capillary porosity is not sufficient to permit a prediction of concrete strength at any given time, because the rate of hydration depends upon the particular cement and curing conditions, as discussed in Chapter 12. The rate of strength gain also depends on the initial *w/c* ratio, with low *w/c* ratio mixes gaining strength more rapidly than high *w/c* ratio mixes, as shown in Figure 13.43 and as discussed in Chapter 19. This is due to the fact that strength depends approximately on the cube of the capillary porosity and because a greater portion of the cement remains unhydrated at low *w/c* ratios. The unhydrated cement particles contribute to the initial strength gain as hydration begins, but do not provide additional strength over time, since they represent portions of cement paste that do not form hydration products. There are two other practical questions that should be discussed:

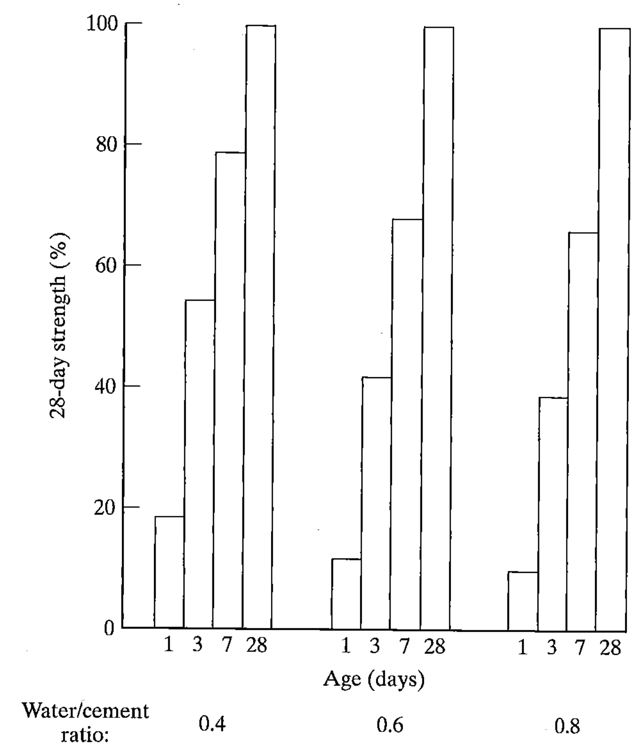


FIGURE 13.43

Relative gain in strength with time of concretes with different water/cement ratios (Type I cement). [From A. Meyer, *Betonstein Zeitung*, Vol. 29, No. 8, pp. 391–394 (1963).]

1. How can the strength at different times be related to the standard 28-day strength?
2. Should the strength developed after 28 days be used for design, since it is known that concrete exposed to water continues to gain strength almost indefinitely?

In practice, it is common to obtain 7-day as well as 28-day compressive strengths. Therefore, it would be very useful to extrapolate 28-day strengths from 7-day (or other) strengths. Of course, this depends upon the cement type and curing temperature. As a general rule, the ratio of 28-day to 7-day strength lies between 1.3 and 1.7, but is usually less than 1.5. Generally, the higher the rate of early strength gain over the first three days, the higher the ratio of 7-day to 28-day strength. Once some experience has been gained for particular mixes, the ratio of strengths may prove to be useful on a particular construction project. As pointed out in Chapter 19, high-strength concrete is often designed for strengths at 56 or 91 days to take advantage of the significant increase in strength that occurs following the first four weeks of curing. The *maturity concept* discussed in Chapter 14, which takes into account both the time and the concrete temperature, is a useful technique for estimating the strength of concrete based on curing conditions.

Cement

The effect of portland cement on concrete strength depends on the chemical composition and fineness of the cement. As discussed in Chapter 3, the strength of hardened cement paste comes primarily from C_3S (earlier strength) and C_2S (later strength), and these effects carry through into concrete. Concretes made with higher C_3S contents gain strength more rapidly, but may end up with slightly lower strengths at later ages. The relative strengths obtained with the five standard types of portland cement are shown in Figure 3.7 (Section 3.2). Although there are considerable differences in strength development up to about one month, at later ages the differences among the five standard types of cement become less important. Cements that hydrate more slowly, whether through changes in composition, curing conditions, or the use of admixtures, tend to develop higher ultimate strengths.

The effects of cement fineness on the strength of concrete are also considerable, since the rate of hydration increases with increasing fineness and leads to a higher rate of strength gain, as indicated in Figure 13.44. Typically, the maximum particle size is about 50 μm, with 10 to 15% less than 5 μm, and perhaps 3% less than 1 μm. Apparently, the fraction of particles less than 3 μm in diameter has the greatest influence on 1-day strengths, while 28-day strengths are most affected by the 3 to 30-μm fraction. However, even though more finely ground cements gain strength more rapidly, very fine grinding should be avoided. With very fine particles, excessive agglomeration may lead to local regions of high *w/c* ratio. On the other hand, there is evidence that particles having diameters greater than about 60 μm contribute little to strength. Over the years, the trend has been toward the production of portland cements with increasing fineness.

It must be recognized that the variability inherent in cement leads to a corresponding variability in concrete strength and therefore requires a higher average concrete design strength (see Chapter 15). Not only do cements that are classified as the same ASTM type vary considerably from plant to plant, but within a given plant, the

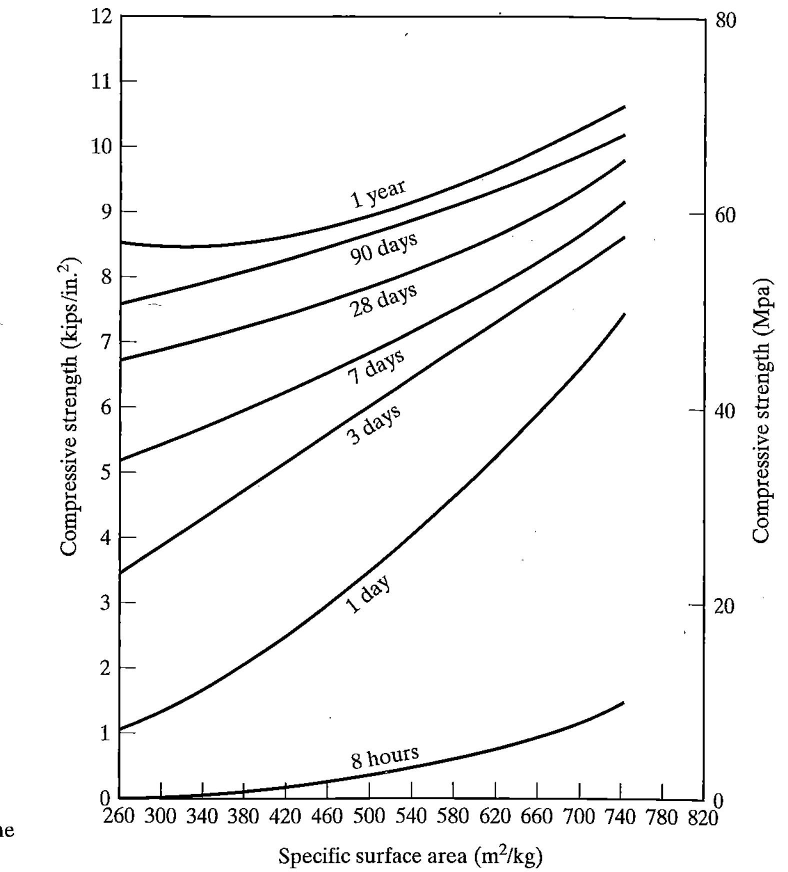


FIGURE 13.44
Effect of cement fineness on strength; $w/c = 0.4$. [Adapted from E. W. Bennett and B. C. Collings, *Proceedings of the Institution of Civil Engineers*, Vol. 43, pp. 443–452 (1969). Reproduced by permission of the Institution of Civil Engineers.]

cement characteristics vary over time, owing to changes in raw materials, burning conditions, etc. It has been estimated that this variability in cement quality leads to a coefficient of variation in concrete strengths on the order of 5%. As described in Chapter 19, the relative effect of cement variability on concrete strength is greater for high-strength concretes than for normal-strength concretes. ASTM C 917 provides a method for evaluating the variability of cement from an individual plant.

Mineral Admixtures

The use of mineral admixtures results in a modification of the structure of hardened cement paste and may result in changes within the interfacial transition zone. For normal-strength concretes, mineral admixtures such as fly ash and blast furnace slag can be used to replace portland cement, with little effect on strength, although, as discussed in Chapter 5, the rate of initial strength gain may be reduced while the long-term strength is increased. The other major pozzolan, silica fume, is generally used to increase strength and will do so even at the same *w/c* ratio. Silica fume not only combines with calcium hydroxide, resulting from the hydration of C_3S and C_2S, but also fills in voids between the cement particles, reducing the flaw size available for crack initiation. The

combined pozzolanic and filler effects of silica fume also increase the strength and greatly decrease the porosity of the interfacial transition zone compared to that obtained in concretes without the admixture. Additional discussion of the effects of mineral admixtures on the strength of concrete can be found in Chapters 5 and 19.

Chemical Admixtures

Chemical admixtures *per se* have very little effect on the strength of concrete, except insofar as they affect the *w/c* ratio or the porosity of concrete. For instance, air-entraining agents principally affect concrete strength due to the increase in porosity, as shown in Figure 13.45. Even in this case, however, the reduction in water content obtainable through the use of entrained air may actually result in an increase in strength for concretes with low cement contents designed for a constant workability. Water-reducing admixtures, especially superplasticizers, may provide an increase in strength, even at the same *w/c* ratio, due to greater cement hydration resulting from improved dispersion of the cement particles and the elimination of large pores that may act as internal flaws.

Chemical admixtures have their greatest effect on the rate of strength gain, either by accelerating or retarding the hydration of cement. In this connection, however, it is worth repeating the observation that a decrease in the initial rate of strength gain of concrete generally leads to somewhat higher long-term strengths, while an increase in the initial rate of strength gain (as obtained with accelerators) invariably results in a decrease in the long-term strength of concrete.

Aggregate

Although the *w/c* ratio is the most important factor affecting strength, the properties of the aggregate cannot be ignored, particularly in the case of tensile and fracture properties. For normal-strength concretes, the aggregate parameters that are most important are the shape, texture, and maximum size of the aggregate. The aggregate strength itself

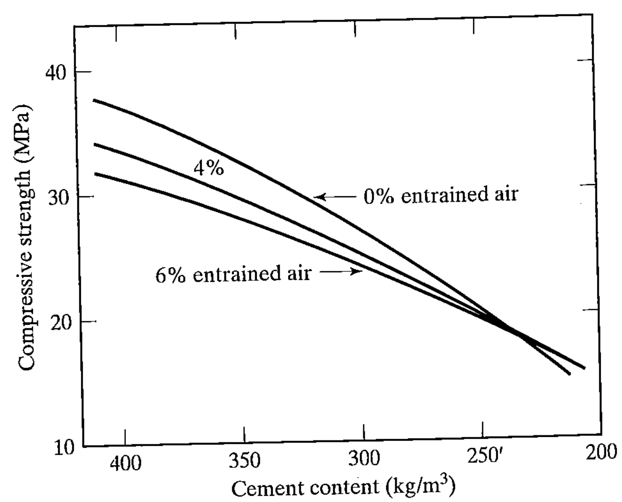


FIGURE 13.45

Relationship between compressive strength and cement content for different quantities of entrained air. [From W. A. Cordon, *Properties, Evaluation, and Control of Engineering Materials*, McGraw–Hill Book Company, New York (1979).]

is of less importance, since aggregates are generally much stronger than the cement paste. In the case of lightweight aggregates (Chapter 20) or high-strength concrete (Chapter 19), however, aggregate strength can play a much greater role due to the relatively high strength of the cement paste constituent of these concretes relative to the strength of the aggregates.

The aggregate texture depends on whether the aggregates are naturally occurring gravels, which tend to be smooth, or crushed rocks, which tend to be rough and angular. Surface texture affects both the bond and the stress at which microcracking begins. The surface texture, therefore, may affect the shape of the stress–strain curve, but has little effect on the compressive strength of the concrete. On the other hand, since surface texture does affect the tensile and flexural strengths of concrete, the ratio between the flexural and compressive strength depends on the type of aggregate, as shown in Figure 13.46. Aggregate type is the dominant factor in fracture behavior for concretes of all strengths, as shown in Figure 19.9 (Section 19.3).

At low *w/c* ratios, crushed rock will lead to higher concrete strengths because of the better mechanical bond, but this effect disappears as the *w/c* ratio increases. However, if mixes are considered on the basis of equal workabilities, this difference becomes unimportant because the lower water requirement of smooth aggregates lowers the *w/c* ratio of the paste, which offsets the effect of lower bond.

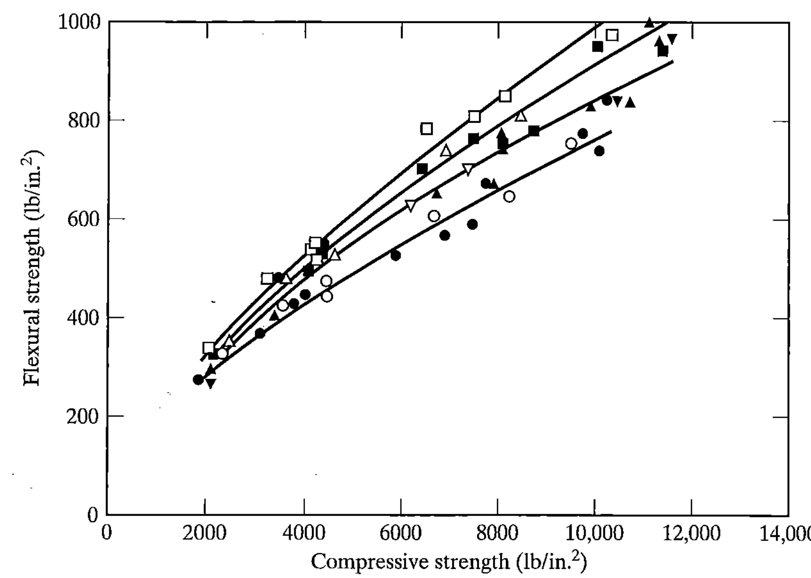


FIGURE 13.46

Relation between flexural strength of concrete beams and compressive strength of equivalent cubes for different types of aggregates. [From R. Jones and M. F. Kaplan, *Magazine of Concrete Research*, Vol. 9, No. 26, pp. 89–94 (1957). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

The use of a larger maximum size of aggregate affects the strength in several ways. Under compressive load, the larger aggregate particles tend to cause a greater stress concentration and a resulting reduction in compressive strength. Also, larger aggregate particles provide more restraint to volume changes in the paste and thus may induce additional stresses in the paste, which may weaken the concrete. The effect of the maximum aggregate size on the strength of concrete at three different *w/c* ratios is shown in Figure 13.47. For the mixes shown in the figure, concrete workability increases as coarse aggregate size is increased. The negative effects of increased coarse aggregate size on strength are offset, however, if mix water is decreased at a constant cement content. As shown in Figure 7.3 (Section 7.1), the result may be such that for constant workabilities, concrete strength may actually increase for mixes with low cement contents, decreasing only for richer mixes. Concretes made with larger aggregates tend to exhibit more variability, probably due to some tendency toward segregation.

In the range of aggregate contents normally encountered, the exact volume of aggregate is only of secondary importance in determining concrete strength. However, as shown in Figure 13.42, major changes in the amount of aggregate relative to the cement content (as well as changes in coarse aggregate size) can have a significant effect on compressive strength. If a constant workability is maintained, however, the strength of concrete is principally dependent on the cement content (Figure 7.3), because of the reduced water requirement as the aggregate size is increased. This is true for both plain and air-entrained concrete. In contrast to compressive strength, an increase in coarse aggregate size and quantity usually results in an increase in tensile strength and fracture energy due to the greater tendency toward crack bridging and the greater path length as cracks pass around the aggregate particles. However, in the case of fracture, these factors are less important than the mechanical properties of the aggregate. Higher strength aggregate results in improved fracture properties (Figure 19.9).

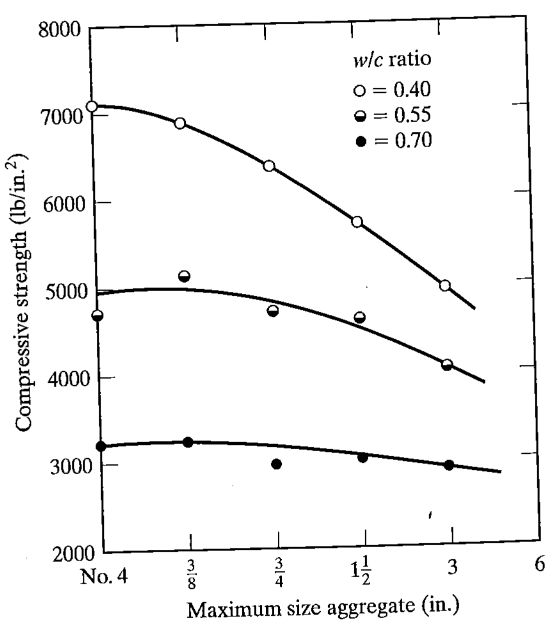


FIGURE 13.47

Effect of maximum size aggregate on concrete strength for three *w/c* ratios. [Adapted from W. A. Cordon and H. A. Gillespie, *Journal of the American Concrete Institute*, Vol. 60, No. 8, pp. 1029–1052 (1963).]

Interfacial Transition Zone

As discussed earlier in this chapter, the strength of the interfacial transition zone has a small, but measurable, effect on the compressive strength of concrete and a major effect on the tensile and fracture properties of concrete. As shown in Figures 13.25a and b, major changes in the strength of the interfacial transition zone can affect compressive strength on the order of 10 to 15%. However, as discussed in Sections 13.1 and 13.3, variations in the strength of the interfacial transition zone may affect the tensile strength on the order of 40%. Increases in aggregate roughness and in the strength of the interfacial transition zone tend to have similar effects on the strength of concrete.

Rate of Loading

Both the tensile and the compressive strengths of concrete increase as the rate of loading increases. The effect on compressive response is shown in Figure 13.8 (Section 13.2). The strain-rate sensitivity of concrete is closely tied to the degree of saturation, with dry concretes showing much less rate sensitivity than wet concretes. The time-dependent behavior of concrete is discussed in detail in Chapter 16.

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PROBLEMS

13.1. How does the structure of the interfacial transition zone (ITZ) differ from the structure of bulk cement paste, as discussed in Chapter 4?

13.2. Calculate the modulus of elasticity of concrete according to the five models given in Table 13.1, assuming Poisson's ratio $\nu = 0.2$, using the following material parameters: $E_p = 2 \times 10^6$ psi, $V_p = 0.25$, $V_a = 0.75$, and $E_a =$ (a) 8×10^6 lb/in.^{2}; (b) 2×10^6 lb/in.^{2}; (c) 0.5×10^6 lb/in.^{2}.

13.3. Calculate the modulus of elasticity of concrete according to the five models given in Table 13.1, assuming Poisson's ratio $\nu = 0.2$, using the following material parameters: $E_p = 20$ GPa, $V_p = 0.20$, $V_a = 0.80$, and $E_a =$ (a) 100 GPa; (b) 20 GPa; (c) 4 GPa.

13.4. Calculate the modulus of elasticity of concrete according to the two models given in Table 13.2, assuming that the material properties are the same as given

in Problem 13.2 and that $V_i = 0.05$ (reducing the value of V_p by the same value), $E_i = 0.5\,E_p$, $\nu_i = 0.2$, and $t_r = 0.01$.

13.5. Calculate the modulus of elasticity of concrete according to the two models given in Table 13.2, assuming that the material properties are the same as given in Problem 13.3 and that $V_i = 0.05$ (reducing the value of V_p by the same value), $E_i = 0.5\,E_p$, $\nu_i = 0.2$, and $t_r = 0.01$.

13.6. If a concrete has a compressive strength of 6000 lb/in.^{2} and a unit weight of 150 lb/ft^3, calculate the modulus of elasticity according to the ACI Building Code.

13.7. If a concrete has a compressive strength of 30 MPa and a density of 1600 kg/m^3, calculate the modulus of elasticity according to the ACI Building Code.

13.8. Discuss the effects of moisture content and the amount and nature of aggregate on the modulus of elasticity of concrete.

13.9. Outline the basic assumptions of the Griffith theory of brittle fracture.

13.10. Describe the failure processes in concrete subjected to compressive loading.

13.11. Would this process be different under pure tensile loading?

13.12. What is the significance of the parameter G_{Ic}?

13.13. Discuss the reasons for the nonlinearity of the concrete stress–strain curve.

13.14. What is the effect of aggregate on the failure of concrete?

13.15. Discuss the statement: "The cement paste–aggregate interface is the weak point of concrete."

13.16. Discuss the nature of damage in concrete under (a) cyclic compression and (b) cyclic tension.

13.17. What factors improve the fatigue life of concrete?

13.18. How does the tensile strength of concrete change when the concrete is subjected to (a) biaxial tension and (b) biaxial tension-compression?

13.19. How does the compressive strength of concrete change when it is placed under (a) biaxial compression and (b) triaxial compression?

13.20. What factors other than the *w/c* ratio affect the strength of concrete?

13.21. Why is it difficult to be sure of the exact *w/c* ratio in concrete?

13.22. How do the relative strengths of concrete at 7 and 28 days vary with the *w/c* ratio?

13.23. For a given cement content and slump, what effect does the addition of entrained air have on the strength of concrete for (a) high cement contents and (b) low cement contents?

CHAPTER 14

Testing of Hardened Concrete

There are a number of reasons for carrying out tests on hardened concrete:

1. On a fundamental level, tests can be used to investigate the physical laws governing the mechanical behavior of concrete. Such tests would be used to help deduce and verify the relationships described in Chapter 13 between the physical and mechanical properties of the constituents of concrete, and the elastic properties and strength characteristics of the concrete made from those materials.
2. Often these physical laws may not be fully developed or understood. In this case, a second type of testing is used, to determine the mechanical properties of a particular type of concrete in some specific application. This is done by simulating the expected conditions of use for a particular concrete as closely as possible and observing the performance of the concrete. These results will not be fundamental in that it will usually not be possible to extrapolate them to predict the behavior of different concretes under different service conditions.
3. On a simpler level, a third type of test would be carried out when the physical laws governing the behavior of concrete are known, but there is a need to evaluate the physical constants, such as E, that occur in these laws.
4. Finally, and most commonly, routine information on the quality of the concrete may be required. This type of testing usually goes under the name *quality control testing*. Here, extreme accuracy of measurement is less important; the speed and ease of carrying out the tests might well govern the choice of test procedures.

Although the tests to be described in this chapter are generally in the category of quality control tests, they are often used for the other three purposes mentioned. Before describing current concrete test methods in detail, however, two other very important concepts must be discussed.

14.1 NEED FOR "STANDARD" TESTS

The strength, durability, and other mechanical properties of concrete should not be considered in any sense as "fundamental" or "intrinsic" material properties. Such variables as

specimen geometry, specimen preparation, moisture content, temperature, loading rate, and type of testing machine and loading fixture will all affect the observed mechanical behavior. Therefore, different test procedures would be expected to yield different results, and experience has shown this to be the case. Thus, when defining some mechanical property, such as compressive strength, it is also necessary to specify the test method used in determining the strength. To complicate the matter further, there are generally no unique relationships between values for mechanical properties measured in different ways. If, for instance, we measure the compressive strength of concretes using two specimen shapes, cylinders and cubes, the ratio (cylinder strength/cube strength) will not have a constant value, but will vary depending on the type of concrete.

Therefore, to try to minimize the confusion that would result if everyone were to use different test procedures, various "standard" test methods have been proposed. For these procedures, the test parameters are rather arbitrarily fixed, so that different people working in different laboratories in different areas can nevertheless carry out comparable tests, and so can generalize their test results. The standard test methods most commonly used in North America are those developed by ASTM, and these will be described in some detail later. However, some agencies, such as the U.S. Bureau of Reclamation, for example, may use somewhat different procedures, and most European countries have their own national standards. In addition, RILEM (The International Union of Testing and Research Laboratories for Materials and Structures) has published test methods that are widely used in Europe. The relevant test designations according to ASTM and Canadian Standards Association (CSA) procedures are given in the Appendix.

It should be noted that as cement and concrete technologies change, the "standard" test methods also change; old test methods are abandoned or modified, and new test methods developed. For instance, it is estimated that about one-third of the ASTM tests are revised annually. Finally, there are properties of concrete (such as impact strength) for which no standard test methods have yet been developed. As such properties become important design characteristics, completely new tests will have to be established for their determination.

14.2 SIGNIFICANCE OF TESTS

It must be emphasized that concrete tests are, in general, carried out *not* on the concrete actually in the structure, but on small "companion" samples of concrete that purport to represent the quality of the concrete in the structure. Unfortunately, this can cause a great deal of confusion. Usually, when we talk about the strength (or other mechanical properties) of concrete, we are referring to the strength of the concrete in the structure at some given age, but what is really measured is the strength of small test specimens. Many studies have shown that there is *not* a particularly good correlation between the strength of concrete defined by ASTM standards and the strength of concrete actually in the structure. This discrepancy arises from differences in the many variables that determine the behavior just mentioned. Although standard tests may well give some indication of concrete quality, many problems have arisen where concrete samples gave acceptable test results while the concrete in the structure was of lower quality and behaved quite differently. Conversely, there have also been cases where perfectly good

concrete structures have been "repaired" on the basis of standard tests giving unacceptable values.

As an example of how the discrepancy between real and measured concrete properties arises, let us examine compressive strength. The concrete in a standard cylinder is subjected to a different compactive effort from the concrete in the structure; the size is different from the size of structural members by several orders of magnitude; the temperature and relative humidity of curing are different; the cylinder remains free of stress until tested, while structural members will be carrying at least some load; under load, the concrete in the structure may be at least partially restrained by steel reinforcement or by other structural elements. In view of this, it would be surprising if the cylinder strengths *did* accurately reflect the strength of the concrete in the structure.

As a result, standard tests carried out on small, supposedly representative, samples do not in any way *guarantee* the quality of the concrete. Rather, concrete design is based on the implicit assumption that if the concrete has been placed, compacted, and cured properly in the structure, and if the test results are satisfactory, there is a very high probability that the material in the structure will behave in an adequate manner. This is quite different from the common assumption that we actually know (or measure) the mechanical properties of the concrete in the structure. Moreover, material properties used in design are obtained from "standard" tests, so that the design procedures used in countries having different concrete test methods must also be somewhat changed, or at least must use different material constants in the design equations.

At this point, one might well ask: If the properties we measure are not necessarily representative of the concrete in the structure, why bother to test the concrete at all? The answer to this question is that there *are* some very good reasons for so doing, but that an accurate measure of the properties of the concrete in the structure is not one of them. The reasons are as follows:

1. Tests can help ensure that the concrete was batched properly; that is, that the proper ingredients, in the proper proportions, were used and that the laboratory mix design was adequate.
2. They can indicate the statistical variability in the properties of the concrete being produced. High variability is one indication of poor concreting practice. (This aspect of quality control will be discussed in detail in Chapter 15.)
3. They may reveal problems arising due to inadvertent changes in materials or environmental conditions.
4. The simple existence of an extensive testing and inspection program will help ensure that the people involved in the production, delivery, and placement of the concrete do not become lax or careless in their operations.
5. If, in spite of all precautions, structural problems do arise, properly carried out and documented test reports will help to pinpoint the problem.
6. Test results may be needed before further construction operations can be carried out. For instance, strength tests may be used as a guide for the time of form removal.

14.3 TESTS FOR COMPRESSIVE STRENGTH

By far the most common test carried out on concrete is the compressive strength test. There are several reasons for this: (1) It is commonly assumed that most of the important properties of concrete are directly related (at least qualitatively) to the compressive strength (though this is often *not* true); (2) since concrete has very little tensile strength, it is used primarily in a compressive mode, and therefore it is the compressive strength that is important in engineering practice; (3) the structural design codes are based mainly on the compressive strength of concrete; and (4) the test is easy and relatively inexpensive to carry out. The compressive strength test and, in particular, the experimental variables that may affect the results are described subsequently in great detail. It should be remembered that the experimental variables discussed in terms of compressive strength will have similar effects on the other mechanical properties of concrete.

ASTM Cylinder Test

As we indicated in the previous section, the test for compressive strength is so sensitive to variations in procedure that it must be carried out strictly according to standard procedures so that results from different testing laboratories (or even within the same laboratory) are comparable. Therefore, the ASTM test is described in some detail. The normal compressive test specimen in North America is a cylinder with a length to diameter (l/d) ratio of 2 : 1, usually of dimensions 150 mm (6 in.) in diameter, 300 mm (12 in.) long. The specimens are prepared and tested as follows:

Molds The ASTM requirements for molds (ASTM C 470) are not restrictive as to the materials that may be used for concrete molds, so long as the molds are nonreactive with concrete, are watertight, and are sufficiently stiff so that they do not deform excessively in use. There are two basic types of molds:

1. *reusable* molds, usually made of heavy-gauge steel or rigid (thick-walled) plastic;
2. *single-use* molds, usually made of formed sheet metal, waxed cardboard, or thin-walled plastic.

In general, the single-use (nonrigid) molds will result in somewhat lower strengths (typically ~3%), with a higher standard deviation. This is in part because the top ends of these molds often take on a somewhat oval shape, and in part because some of the applied compactive effort (rodding or vibration) goes into deforming the mold rather than consolidating the concrete. While the somewhat lower strengths obtained with the single-use molds are not of much importance for normal concretes, they can be significant for very high strength (>100 MPa) concretes.

Molding the Specimens (ASTM C 31) The mold is placed on a firm, level surface, free from vibration. From a representative sample of concrete (as discussed in Chapter 15), the mold is filled in layers, depending on the method of consolidation. If the slump is more than 75 mm (3 in.), the concrete is consolidated by rodding; if the slump is less than 25 mm (1 in.), the concrete is consolidated by vibration. For slumps between 25 and 75 mm, the specimens may be consolidated either way. (The specifications for a particular job may require a certain method of consolidation, which would supersede the foregoing guidelines.) The reason for these alternative methods is that a poorly compacted cylinder will have a lower strength than will a properly compacted one. For stiff mixes, which will always be compacted by vibration in the field, rodding simply will not compact the cylinders sufficiently, while vibration may cause segregation of wet mixes.

If the specimen is to be rodded, it should be filled in three equal layers. Each layer is rodded 25 times with a 16-mm (5/8-in.) diameter steel rod with a rounded end. For the upper layers, the rod should penetrate about 25 mm (1 in.) into the underlying layer. After rodding each layer, the mold is tapped lightly 10 to 15 times with a mallet (or open hand for light-gauge molds) to close any holes left after rodding. Specimens to be vibrated are filled in two equal layers; either internal or external vibration may be used. After consolidation, the top surface is finished by striking off with the tamping rod or with a trowel.

Curing Cylinders are made primarily for three reasons: (1) to check whether the laboratory-designed mix achieves the specified design strength, as a basis for acceptance or for quality control; (2) to determine when forms can safely be removed; and (3) to determine when a structure may be put into service, or when prestressed concrete members may be tensioned. Cylinders are also used for comparison with results from in-place tests and to determine the adequacy of field curing and protection of the concrete. The curing conditions for the cylinders depend on the purpose for which they were made.

If the cylinders are made for reason 1, they must be stored at between 16 and 27°C (60 and 80°F) for the first 24 to 48 h, in such a way that moisture loss is prevented. The specimens are then removed from the molds and stored in a standard moist room or in saturated lime water at 23 ± 2°C (73 ± 3°F) until tested. If the cylinders are cast in a laboratory (ASTM C 192), they must be maintained at 23 ± 2°C from the moment of casting. If the cylinders are made for the estimation of form-removal time, or to determine the adequacy of protection of the concrete, they are stored as near to the part of the structure in question as possible and should be protected from the elements in the same way as the structure. These specimens should be tested before the forms are removed. For specimens used to determine when a structure may be put into service, or for comparison with in-place test results, the cylinder molds should be removed at the same time that the forms are removed, so that the moisture condition of the specimen approximates the moisture condition of the structure. In some situations, in order to match the moisture conditions, only the specimen ends should be exposed.

Capping the Cylinders Cylinders cast as just described will have end surfaces (particularly the top surface) that are rough and not necessarily plane or parallel. If tested in this condition, the apparent strength of the concrete would be considerably reduced as a result of stress concentrations that are introduced on loading. Convex ends will lead to lower measured strengths than concave ones, because the stress concentrations introduced are more severe. It is required that the specimen ends must be plane within

0.050 mm (0.002 in.). One way to achieve planeness is by grinding the ends; this is satisfactory, but is expensive and time consuming. It is used mostly with high-strength concretes, though some laboratories use this technique routinely for all concretes. The most common way of achieving this planeness requirement is to *cap* the ends of the cylinders (ASTM C 617) with a suitable material. Three different capping materials are permitted. A thin layer of stiff portland cement paste may be used on freshly molded specimens. On hardened cylinders, either high-strength gypsum plaster or sulfur mortar may be used, the latter being much more common. The sulfur mortar is prepared by heating it to about 130°C (265°F) and then pouring it into lightly oiled steel capping plates. The cylinders are set into the sulfur in such a way that the caps end up being about 3-mm (1/8-in.) thick and are aligned so that the deviation of each cap from perpendicularity with the axis of the specimen is less than 0.5°. After capping, the cylinders should be kept moist until tested; commonly, cylinders are tested soon after capping. (It should be noted that sulfur loses strength and pourability with use, and therefore it is required that the oldest material in a sulfur pot should not have been used more than five times.)

For concretes with strengths up 50 MPa (7000 lb/in.^{2}), the strength of the capping material should be at least 35 MPa (5000 lb/in.^{2}). For higher-strength concretes, the compressive strength of the capping material should be at least as high as the concrete cylinder strength (or produce a strength that is at least 98% of the strength of a compression cylinder capped with neat cement paste or ground plane). However, capped ends will generally result in a higher standard deviation than ground ends.

An alternate technique for end preparation gets away from the necessity of either capping or grinding. This technique (ASTM C 1231) involves the use of *unbonded caps*. Basically, an elastomeric pad is placed within a metal retaining ring, and the assembly is then placed over the specimen end. The pad conforms to the shape of the cylinder end, but is prevented from spreading laterally by the metal retaining ring. This provides a uniform load across the specimen ends. The pads may be used a maximum of 100 times. The strengths obtained are slightly lower (<2%) than the strengths obtained using capping or grinding. At the present time, this system is usually limited to concretes with strengths of 50 MPa (7000 lb/in.^{2}) or less and is prohibited for concretes with strengths in excess of 80 MPa (12,000 lb/in.^{2}).

Determination of Compressive Strength (ASTM C 39) Once the specimens have been capped, they are ready for testing. The tests may be carried out in any suitable testing machine. Two hardened bearing blocks are used, a solid one that the specimen sits on and a spherically seated one that will bear on the upper surface. These blocks must be plane to within 0.025 mm (0.001 in.). For 150-mm (6-in.) diameter cylinders, the maximum diameter of the spherically seated block is 255 mm (10 in.). Care must be taken to ensure that the ratio of the maximum diameter of the bearing block to that of the specimen does not exceed the values given in ASTM C 39; a bearing block that is too large will generally lead to compressive strength values that are too high. Because strength is dependent on the loading rate, the specimen is then loaded at a controlled rate of 0.15 to 0.35 MPa/s (20 to 50 lb/in.^{2}/s) for hydraulic machines, or at a deformation rate of 1 mm/min (0.05 in./min) for mechanical machines, until failure, which is defined as the maximum load the specimen can carry. The maximum load and type of failure are then reported.

Cube Tests

Cube tests are used as the normal compressive test in Great Britain, Germany, and some other parts of Europe. The British Standard requires a 150-mm (6-in.) cubic mold, which is filled in three layers, each layer being rodded 35 times with a 25-mm square rod; alternatively, the cube may be compacted by vibration. It is then handled in much the same way as the cylinders described, except that the storage temperature is 20 ± 1°C (68 ± 2°F). The cube is tested at right angles to the position at which it is cast; this means that the faces of the cube in contact with the bearing platens were cast against the sides of a rigid steel mold. Therefore, the bearing faces are sufficiently plane as to require no capping or grinding. This is the chief advantage of the cube specimen. The specimen is then loaded at a rate of 0.25 MPa/s (33 lb/in.^{2}/s) to failure.

The main drawback to the cube test is that the friction between the platens and the specimen ends creates much more *confinement* (triaxial compression) in the specimen than is the case with the larger cylindrical specimens (see sections that follow). This leads to higher strength values when measured on cubes rather than cylinders. However, though the ratio between cube strengths and cylinder strengths is commonly assumed to be 1.25, the ratio is not, in fact, a constant one, ranging from about 1.3 for low-strength concretes to about 1.04 for higher strength concretes, as shown in Figure 14.1.

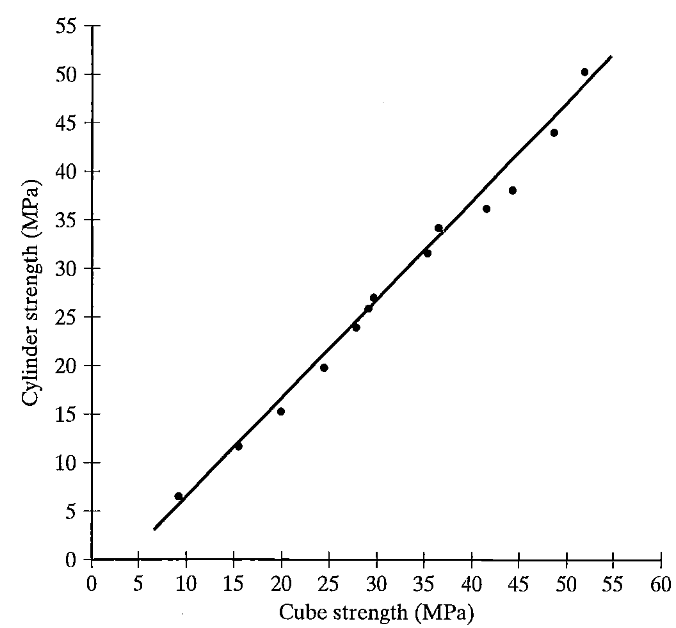


FIGURE 14.1

Relationship between cylinder strength and cube strength. [Data taken from R. H. Evans, "The Plastic Theories for the Ultimate Strength of Reinforced Concrete Beams," *J. Inst. Civil Engineers*, Vol. 21, pp. 120–138 (1943–44).]

Factors Affecting the Measured Compressive Strength

The compressive strength tests just described appear to be perfectly straightforward. Unfortunately, the results obtained can be affected considerably by a number of factors. If we are to be able to interpret the strength values obtained by the standard (although arbitrary) procedures, it is necessary to examine in detail how compressive strength is affected by the test parameters.

Stress Distribution in Specimens The compression test assumes a state of pure, uniaxial compression. However, this is not really the case, because of friction between the ends of the specimens and the platens. This frictional force arises from the fact that, because of the differences in the modulus of elasticity and Poisson's ratio for steel and concrete, the lateral strain in the platens is considerably less than the lateral expansion of the ends of the specimen *if they were free to move*. Thus, through friction, the platens act to restrain the lateral expansion of the ends of the specimens and to introduce a lateral confining pressure near the specimen ends. This confining pressure (which also introduces shear stresses) is greatest right at the specimen end and gradually dies out at a distance from each end of approximately $(\sqrt{3}/2)\,d$, where d is the specimen diameter. (The exact nature of the stress distribution depends on the type of contact between the specimen and the platens for a particular test.) The manifestation of this lateral confining pressure is often the appearance of relatively undamaged cones (or pyramids) of concrete in specimens tested to failure, as shown in Figure 14.2a. Thus, for a standard cylinder with $l/d = 2.0$, only a small central portion of the cylinder is in true uniaxial

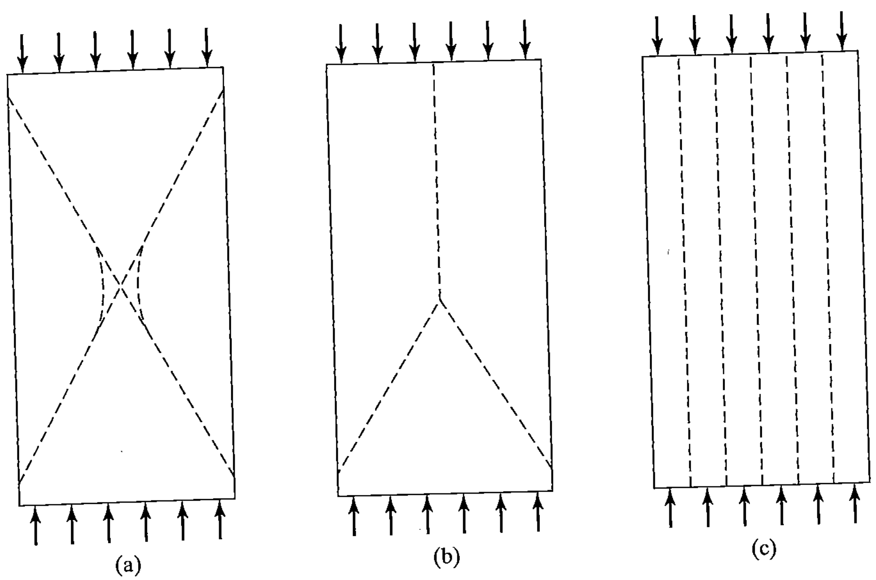


FIGURE 14.2

Typical failure patterns for concrete cylinders in compression: (a) confinement at both ends; (b) confinement at one end and splitting failure at the other; (c) splitting failure.

compression, the remainder being in a state of triaxial stress. The effect of this type of end restraint is to give an apparently higher strength than the "true" compressive strength of the specimen.

The apparent strength will increase as the relative volume of the specimen subjected to lateral restraint increases. This accounts for the apparently higher strengths of cylinders with l/d ratios less than 2.0. It also accounts for the fact that, in general, cubes indicate higher strengths than cylinders; they simply are not long enough for the effects of the end restraint to die out. For a given specimen geometry, higher confining stresses will also result in higher apparent strengths (Chapter 13). Conversely, it should be noted that if a very soft material is placed between the ends of the specimen and the platens to eliminate friction, the outward flow of the material under pressure may induce lateral *tensile* forces in the ends of the specimen. This will reduce the strength, and the specimens may fail by vertical splitting. It is the usual case, however, that friction-reducing materials are unable to completely eliminate the effects of confinement.

On a more fundamental level, to speak at all of a "compression" failure of concrete (or most other materials) is incorrect. Since compression tends to squeeze atoms and molecules closer together, it is hard to see how pure compression can lead to failure. In a compression test, however, there are also secondary tensile stresses induced in the specimen, at right angles to the axis of the specimen. Since concrete is relatively weak in tension, it is these stresses that cause cracking and failure. It has been suggested that the failure of concrete in tension will occur at limiting strains of about 0.0001 to 0.0002. For ordinary concrete, with Poisson's ratio approximately equal to 0.2, such lateral tensile strains will occur at fairly low compressive loads, and this could be the cause of failure, as shown in Figure 14.2c. This is probably the natural mode of failure in pure compression. The stresses induced due to end restraint may cause an apparent conical failure of the specimen, as indicated in Figure 14.2a.

Since we cannot avoid some end restraint, it is likely that failure occurs through some combination of forces, as indicated schematically in Figure 14.2b. Tensile cracks may not be able to propagate through the portions of the specimen under a lateral confining stress.

Test Apparatus As well as being plane, the ends of the specimen should also be perpendicular to the axis of the specimen. Since this is very difficult to achieve exactly, a spherically seated platen is used; a small deviation from parallelism will then not affect the strength. ASTM C 39 specifies that the spherical head should be lubricated with conventional oil (rather than pressure-type grease), so that once the head is in full contact with the specimen, further tilting will not occur. Changing the degree of lubrication in the spherical seat can cause apparent changes in strength.

Different types of platens can also lead to different results. Since even steel platens will distort slightly under load, platens of different stiffness will lead to somewhat different stress distributions across the ends of the specimens.

The type of testing head that probably best approximates true uniaxial compression is the so-called "brush platen." This consists of filaments about 5×3 mm (0.20×0.12 in.) in cross section, with gaps of about 0.2 mm (0.008 in.) between them. This allows the concrete to expand laterally with very little restraint. Such platens will, of course, give lower strengths than ordinary steel platens.

Testing machines themselves can vary in rigidity, from relatively hard to relatively soft. This difference has been found to have only a negligible effect on the measured compressive strengths. However, very rigid machines will sometime permit the measurement of at least part of the *descending* branch of the load vs. deflection curve.

Effect of *l/d* Ratio The standard cylinder has a length-to-diameter ratio of 2.0, even though, as we have seen, this still produces considerable lateral confinement within the specimen. It has been found that if other *l/d* ratios are used, the specimen strength changes due to the end effects described. The general effect of the *l/d* ratio on strength is shown in Figure 14.3. However, the exact shape of the curve depends to some extent on the end conditions and on the type of concrete tested. It has been found that high-strength concrete is less affected by variations in specimen geometry. As a general rule, for specimens subjected to end restraint, an *l/d* ratio of 3.0 is high enough to provide true uniaxial compression (in the center portion of the specimen) without causing problems due to slenderness. The effect of the *l/d* ratio is particularly significant when tests are carried out on drilled cores, where, out of necessity, the *l/d* ratio is often less than 2.0. In this case, correction factors (Table 14.1) are provided (ASTM C 42) so that specimens with different *l/d* ratios can be compared on the same basis. If the *l/d* is greater than 2.0, the specimen can be cut to size before testing.

Specimen Geometry Since different geometries of specimens—cylinders or cubes—give different values of strength, which specimen shape, then, is "best"? Clearly, cubes are somewhat easier to test, since they require no capping. However, the tendency is to prefer cylinders to cubes, especially in research, as there is a feeling that they better represent the strength of the concrete. But as we have seen, small concrete specimens do not particularly represent the strength of the concrete in the structure; they give only comparative data. It thus makes little difference which specimen geometry is chosen, as long as it is used consistently.

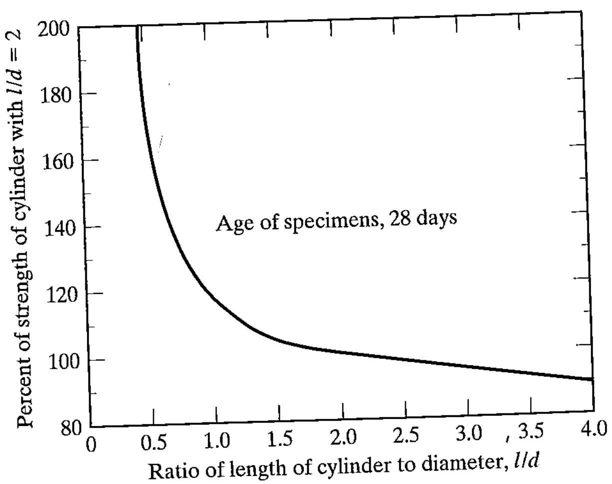


FIGURE 14.3

Relation of length and diameter of specimen to compressive strength. [From W. H. Price, *Journal of the American Concrete Institute*, Vol. 47, No. 6, pp. 417–432 (1951).]

TABLE 14.1 Relationship between *l/d* Ratio and Compressive Strength[a]

l/d Ratio	*Strength Correction Factors*
2.00	1.00
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87

[a]From ASTM C 42. Copyright ASTM INTERNATIONAL. Reprinted with permission.

Both the strength and the variability in strength of concrete decrease as the specimen size increases. This is generally explained by the "weakest link" theory. That is, we assume that the strength of a concrete specimen is governed by the weakest element ("link") within it; the larger the size of the specimen, the more likely it is to contain an element that will fail at a (given) low load. Such points of weakness are, in general, the types of flaws that were discussed in Chapter 13. However, as specimens become larger, the differences in the distributions of flaws within each sample become less, and hence the variability decreases. Figure 14.4 shows the variation in strength for cylinders of $l/d = 2.0$, but with different diameters.

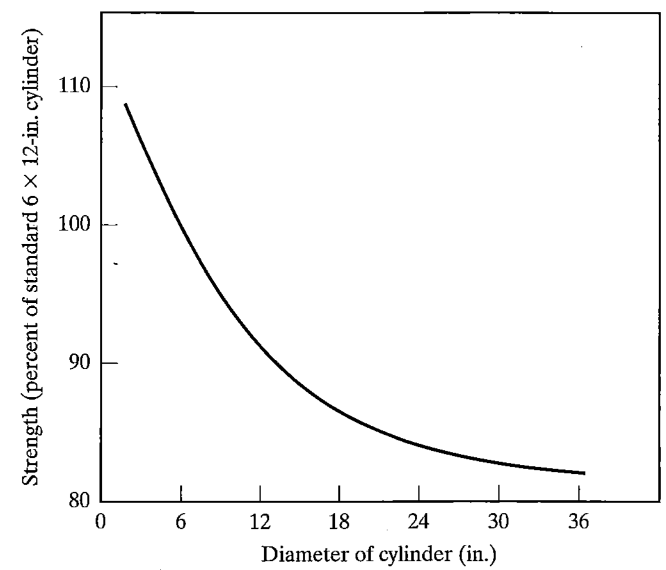


FIGURE 14.4

Effect of cylinder size on compressive strength of concrete, with $l/d = 2.0$. [From W. H. Price, *Journal of the American Concrete Institute*, Vol. 47, No. 6, pp. 417–432 (1951).]

Rate of Loading In general, the higher the rate of loading, the higher the measured strength (Section 16.3). The reasons for this are not completely clear. It may be that slow rates of loading allow more subcritical crack growth to occur, thus leading to the formation of larger flaws and hence smaller fracture loads (Chapter 13). On the other hand, it may be that slower loading rates allow more creep to occur, which will increase the amount of strain at a given load. When the limiting value of strain is reached, failure will occur. More likely, the observed rate of loading effect is due to a combination of these, and perhaps other factors as well. The extreme case of static fatigue is discussed in Chapter 16. Clearly, to obtain comparable results, the rate of loading in practice must be maintained within fairly narrow limits.

Moisture Content Most concrete specifications, such as ASTM C 39, described earlier, require that concrete be maintained and tested in a saturated state. It has been found that concrete that has been dried shows an increase in strength. The reasons for this are not completely understood; it may have something to do with the change in the structure of the C–S–H on drying, or it may simply represent a change in the internal friction and cohesion on a macroscopic scale; that is, moisture may have a "lubricating" effect, allowing particles to slip by each other in shear more easily. The lower compressive strength of wet concrete may also be due to the development of internal pore pressure as a load is applied. For an oven-dried specimen, the increase in strength is on the order of 10 to 15%. This increase in strength appears to be reversible, as subsequent resaturation will return the concrete to its original saturated strength.

However, the experimental evidence is somewhat contradictory, particularly for tensile and flexural strengths. Since concrete has a fairly low diffusion rate, and dries only from the outside, it is very difficult to get perfectly "dry" concrete, or to fully resaturate concrete. Thus, the degree of dryness depends on the size and shape of the specimen. Drying too quickly may also induce tensile cracks due to nonuniform drying (and hence differences in drying shrinkage) of the specimen. The cracks do not have much effect on the compressive strength, but will lower the apparent flexural and tensile strengths. Slow drying, on the other hand, where cracking is prevented, will increase the flexural and tensile strengths. Similar effects are sometimes noted when dry specimens are resaturated. The effect of the moisture content on strength becomes an important consideration when testing drilled cores.

To take the moisture content of the specimens into account, ASTM C 42 recommends that concrete cores be conditioned to a moisture state most representative of the state of the in-place concrete. However, drying of the cores as specified will most commonly lead to specimens with a moisture gradient from the outside to the center. This will lead to lower measured strengths than those obtained on specimens with a uniform moisture content.

Temperature at Testing For reasons that are also not fully understood, the temperature of the specimen at the time of testing will affect the strength, as shown in Figure 14.5. Higher test temperatures will result in lower strengths, even for concretes that were identically cured in standard conditions. It is likely that at least part of the effect is due to loss of moisture from the specimen while being conditioned to the higher temperatures.

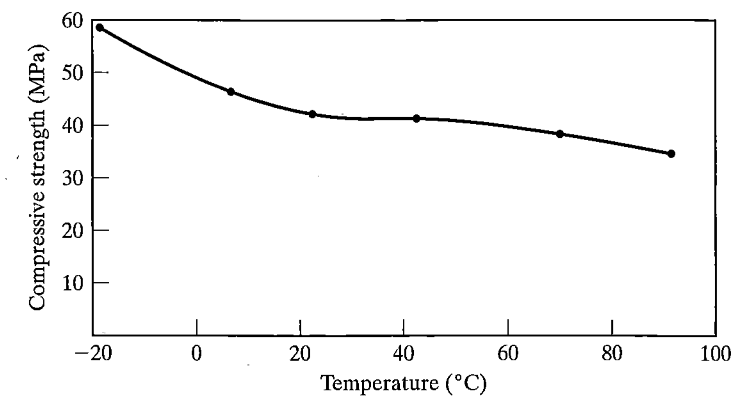


FIGURE 14.5

Compressive strength as a function of temperature at the time of testing. [From J. C. Saemann and G. W. Washa, *Journal of the American Concrete Institute*, Vol. 54, No. 5, pp. 385–389 (1957).]

Interpretation of Results From the preceding few pages, it should be apparent that the compressive strength is neither a fundamental nor a uniquely defined property of concrete. Its value depends very markedly on the method of measurement. Thus, since compressive strength results are of value primarily on a comparative basis, there must be strict adherence to a standardized test method.

14.4 OTHER CONCRETE TESTS

Tensile Strength

No standard tests have yet been adopted by ASTM to provide a direct measurement of the tensile strength of concrete. The problem of secondary stresses induced through gripping makes the test results difficult either to interpret or to reproduce. However, RILEM has prepared a recommendation for a direct tension test of concrete, designed primarily for research rather than for routine control. This method involves applying direct tension to either cylindrical specimens or prismatic specimens (with a square cross section) through end plates glued to the concrete. The ends of the specimen must be sawed off to remove end effects due to casting or vibration; they must be perpendicular to the axis of the specimen within 1/4°. Further, the ends must be carefully cleaned so that the glue (typically a polyepoxy resin) adheres uniformly to the entire surface. The load is applied at a rate of 0.05 MPa/s (7 lb/in.^{2}/s) until failure occurs. As yet, there is not enough experience with this test method for a proper assessment of its

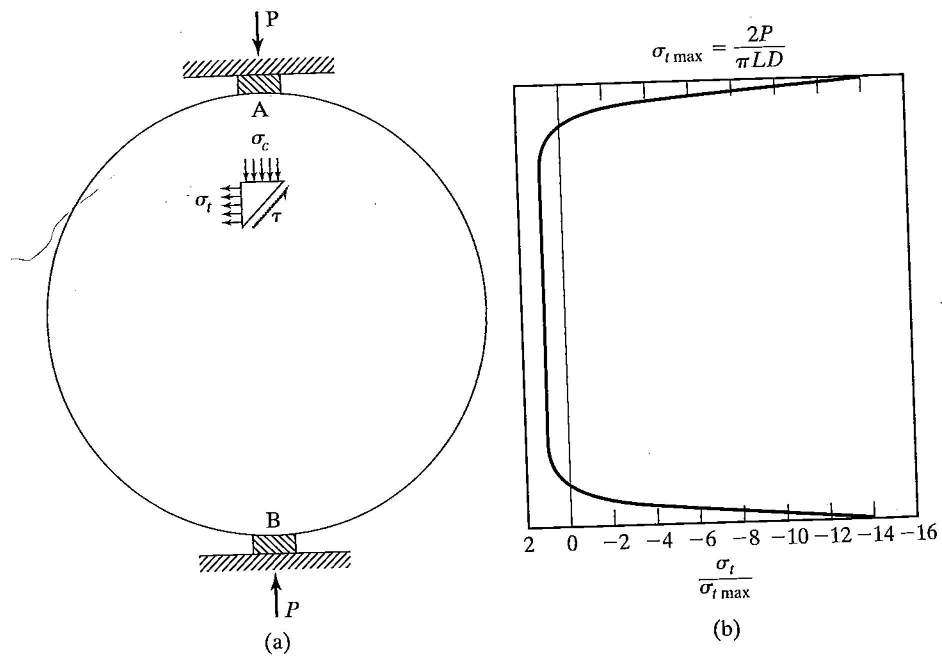


FIGURE 14.6
Stress distribution across loaded diameter for a cylinder compressed between two flat plates.

usefulness to be made. The U.S. Bureau of Reclamation also specifies a direct tension test (Procedure 49A) that uses bonded end plates.

There is, however, one common method for estimating the tensile strength of concrete, through an indirect tension test. The *splitting tensile test* is carried out on a standard cylinder, tested on its side in diametral compression, as shown in Figure 14.6a. The element on the vertical diameter shown in Figure 14.6a is subjected to the following stresses:

$$\text{vertical compression } \sigma_c = \frac{2P}{\pi LD}\left[\frac{D^2}{r(D-r)} - 1\right] \quad (14.1)$$

$$\text{horizontal tension } \sigma_t = \frac{2P}{\pi LD} \quad (14.2)$$

where P is the applied compressive load, L the cylinder length, D the cylinder diameter, and r the distance of the element from the top of the cylinder.

It is not practical to apply a true "line" load along the top and bottom of the specimen, partly because the sides are not smooth enough, and partly because this would induce extremely high compressive stresses near the points of load application. Therefore, the load is usually applied through a narrow bearing strip of relatively soft material. The tensile stress distribution along the vertical diameter of the specimen is then as shown in Figure 14.6b. That is, there are very high compressive stresses near the ends of

the vertical diameter, and a nearly uniform tensile stress acting over about the middle two-thirds of the specimen. Since the concrete is much weaker in tension than in compression, failure will be in splitting tension at a much lower load than would be required to crush the specimen in compression, thus permitting an estimate to be made of the tensile strength of the concrete.

As specified in ASTM C 496, the test is carried out on cylindrical specimens, either cast cylinders or drilled cores. The bearing strips are made from 3-mm (1/8-in.) plywood that is free of imperfections and is about 25-mm (1-in.) wide. The specimen is aligned in the machine, and the load is applied at a rate of 690 to 1380 kPa/min (100 to 200 lb/in.^{2}/min) splitting tensile stress until the specimen fails. The splitting tensile strength is then calculated from Eq. (14.2). It should be noted that it is also possible to carry out splitting tension tests on cubes. This is done by loading through two hemispherical bars along the center lines of two opposite faces. This gives much the same results as testing a cylinder; the horizontal tensile stress is

$$\sigma_t = \frac{2P}{\pi a^2} \quad (14.3)$$

where a is the side of the cube.

There is no simple relationship between the values of tensile strength found from the splitting test and those measured in direct tension. While it is commonly assumed that splitting tensile strength values are 5 to 12% higher than direct tensile strength values, this is not always true. Some recent studies have shown very little difference between the two values of tensile strength, and for some concretes the direct tensile strengths have been found to be slightly higher than the splitting strengths. It would appear that the ratio of direct tensile strength to splitting tensile strength depends upon the strength level of the concrete and upon the maximum aggregate size. It should also be noted that recent more detailed analyses of the splitting tension test have shown that the splitting tensile strength determined according to these test procedures should not be considered to be a "true" material property. The results obtained depend upon the specimen size and the width and type of the load bearing strips.

Flexural Strength

The flexural strength of concrete may be determined following the procedure of ASTM C 78. The specimen is a beam 150 × 150 × 500 mm (6 × 6 × 20 in.). The mold is filled in two equal layers, each layer being rodded 60 times, once for each 13 cm^2 (2 in.^{2}) of top surface area. Stiffer mixes may be consolidated by vibration, as specified for compression cylinders. The beams are cured in the standard manner and are then tested in flexure in third-point loading, as shown in Figure 14.7. Tests may also be carried out on beams sawn from hardened concrete. The cast specimens are tested turned on their sides with respect to their position as molded. This should provide smooth, plane, and parallel faces for loading. If for some reason this is not the case, the specimens must be ground, capped, or shimmed. Specimens are loaded at a rate of 860 to 1200 kPa/min (125 to 175 lb/in.^{2}/min). The theoretical maximum tensile strength, or *modulus of rupture, R* (or f_r'), is then calculated from the simple beam bending formula for third-point loading, namely,

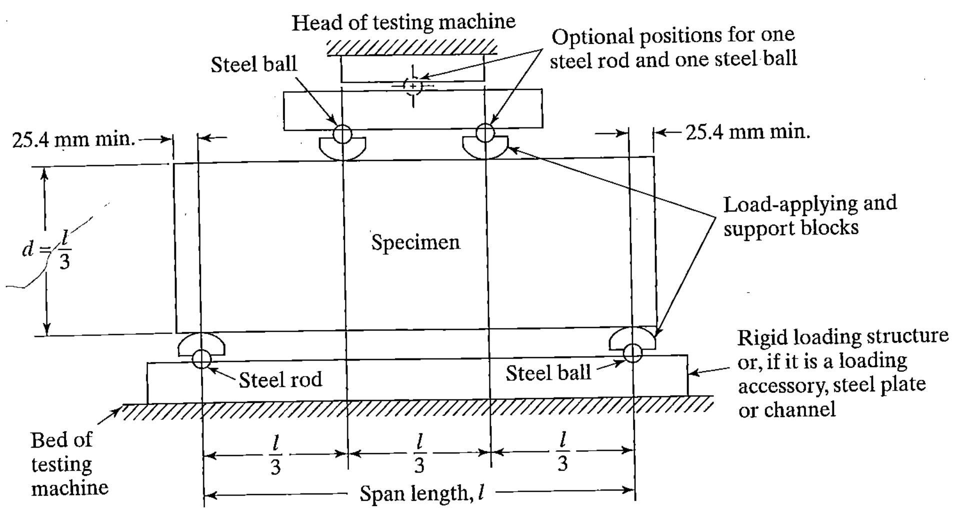


FIGURE 14.7

Diagrammatic view of a suitable apparatus for flexure test of concrete by third-point-loading method. This apparatus may be used inverted. If the testing machine applies force through a spherically seated head, the center pivot may be omitted, provided that one load-applying block pivots on a rod and the other on a ball. (From ASTM C 78. Copyright ASTM INTERNATIONAL. Reprinted with permission.)

$$R = \frac{Pl}{bd^2} \tag{14.4}$$

where P is the maximum total load indicated, l the span length, b the specimen width, and d the specimen depth. Equation (14.4) holds only if the beam breaks between the two interior loading points (i.e., in the middle third of the beam). If the beam breaks outside these points by not more than 5% of the span length, Eq. (14.4) is replaced by

$$R = \frac{3Pa}{bd^2} \tag{14.5}$$

where a is the average distance between the point of fracture and the nearest support. The results of tests where failure occurs even closer to the supports are discarded.

This test tends to overestimate the "true" tensile strength by about 50%, largely due to the fact that the simple flexure formula [Eq. (14.4)] assumes that the stress varies linearly across the cross section of the beam. But because concrete has a nonlinear stress–strain curve, this assumption is not true. Near failure, particularly, the stress block is more nearly parabolic than triangular, as indicated in Figure 14.8. However, the test remains very useful, since concrete members tend to be loaded in bending rather than in axial tension, and thus the values obtained from a flexure test are a better representation of the concrete property that is of interest. This test is most widely used for quality control of highways and airport runways, where it gives more useful information than do compression tests.

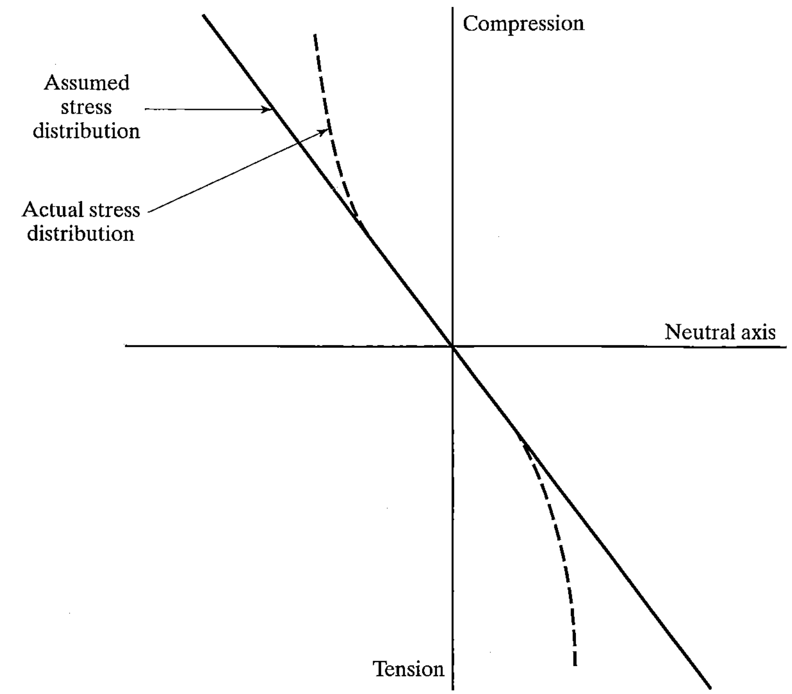


FIGURE 14.8

Stress distribution across the depth of a concrete specimen in flexure.

It is also possible to carry out flexure tests in center-point loading as described in ASTM C 293. This test method is not as good as third-point loading and is not a substitute for it. The difficulty is that, ideally, in the third-point loading test the specimen, in the middle third of the span, is subjected to a pure moment, with zero shear. In the center-point test, there are substantial shear forces as well as unknown stress concentrations at the point of load application, which act along the line on which the specimen generally fails. Center-point loading yields higher strengths than third-point loading.

As in the case of the compression test, the test parameters can greatly affect the observed strengths. In particular, the size effect is very important. One additional reason for the fact that flexure tests give higher values than a direct tension test is that in direct tension, the total volume of the specimen is stressed, while in flexure, only a relatively small volume of material near the bottom of the beam is subjected to high stresses. Thus, if we believe the "weakest link" theory, the likelihood of finding a sufficiently weak element of concrete is reduced in flexure. This also contributes to the observation that center-point loading yields higher strength values than does third-point loading. If the load points are moved still farther apart, the strength continues to decrease. The effects of specimen size are shown in Figure 14.9. However, as the specimen size increases, the coefficient of variation decreases, which is why flexural strengths determined by third-point loading show less scatter than when center-point loading is used. Similarly, the apparent flexural strength increases as the rate of loading increases (Section 16.3), as

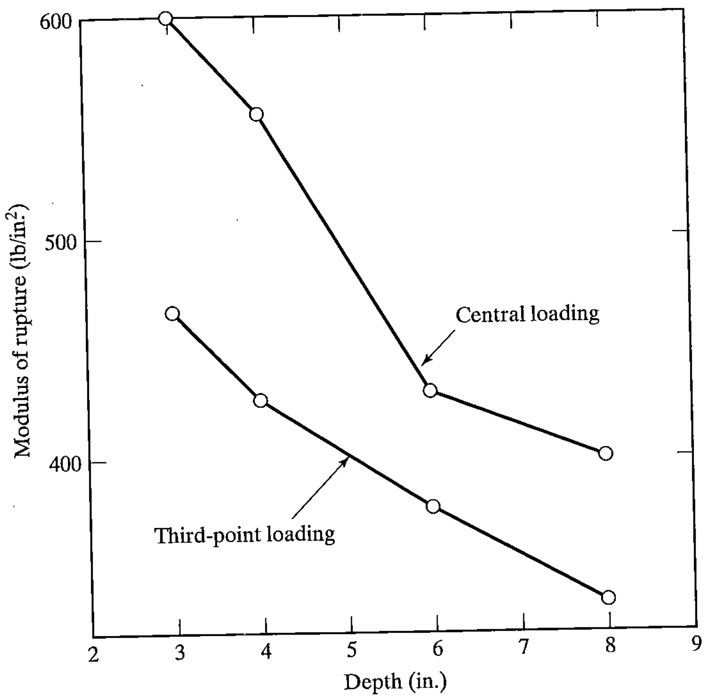


FIGURE 14.9

Effect of depth of specimen on the modulus of rupture of concrete. [From P. J. F. Wright, *Magazine of Concrete Research*, Vol. 4, No. 11, pp. 67–76 (1952). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

found in compression tests. Temperature effects in flexure are also similar to those described earlier.

Modulus of Elasticity

As we have seen in detail in Chapter 13, the stress–strain curve for concrete is not linear. However, to calculate the stiffness or expected deflection of structural members, it is necessary that some estimation of the modulus of elasticity, E, be obtained. Although it is possible to define E in a variety of ways for a material with a nonlinear stress–strain curve (see Section 13.2), the most common way for concrete is to measure the *chord modulus of elasticity*, E_c, following the method outlined in ASTM C 469. A standard cylindrical specimen to which a compressometer has been attached is first loaded and unloaded, primarily to properly seat the strain gauges. It is then loaded slowly (at a stress rate of 241 ± 34 kPa/s or 35 ± 5 lb/in.²/s) in compression, and a stress–strain curve obtained. The chord modulus of elasticity is calculated using the expression

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - 0.000050} \tag{14.6}$$

where S_2 is the stress corresponding to 40% of the ultimate load; S_1 the stress corresponding to a longitudinal strain, ϵ_1, of 50×10^{-6}; and ϵ_2 the longitudinal strain produced by stress S_2. From the same test, Poisson's ratio, ν, can be calculated as

$$\nu = \frac{\epsilon_{t_2} - \epsilon_{t_1}}{\epsilon_2 - 0.000050} \tag{14.7}$$

where ϵ_{t_2} and ϵ_{t_1} are the transverse strains at midheight of the specimen produced by stresses S_2 and S_1, respectively.

The modulus of elasticity in shear, G, is not obtained by direct measurement. Rather, it is calculated from the elastic relationship

$$G = \frac{E}{2(1 + \nu)} \tag{14.8}$$

Dynamic Modulus of Elasticity

It is often desirable to have some nondestructive test for the stiffness of a concrete specimen so that progressive changes in the material due to sustained chemical attack, repeated freeze–thaw cycles, aging, or other factors can be measured. For this purpose, dynamic modulus of elasticity measurements are very convenient, since they are nondestructive and since there is also a general (if empirical) relationship between the modulus of elasticity and the compressive strength.

The standard way of carrying out these tests is to measure the dynamic modulus of elasticity, E_d, by vibration of concrete specimens at their natural frequency. The method is described in detail in ASTM C 215, using one of the test setups shown schematically in Figure 14.10. Basically, a test specimen (either a cylinder or a prism) is supported at its nodal point (or points) so that it may undergo free–free vibration without significant restriction (i.e., for transverse vibration, it is supported at 0.224 of the length from each end of the specimen; for longitudinal or torsional vibration, it is supported at the center). The driver or impactor and pickup unit are placed as shown in Figure 14.10c. Using the *forced resonance test* (Figure 14.10a), the specimen is then forced to vibrate at various frequencies; the frequency of vibration that gives the maximum output indicates the fundamental frequency. Using the *impact resonance test* (Figure 14.10b), the specimen is struck and its response is monitored using a lightweight accelerometer; the fundamental frequency is measured using a waveform analyzer or frequency counter. For comparison, all specimens should be analyzed using the same test method.

E_d can be calculated from either the transverse or longitudinal frequency, using the general expression

$$E_d = KMn^2 \quad \text{MPa} \tag{14.9}$$

where M is the mass of specimen (kg), n the fundamental frequency of vibration (Hz), and K a constant that depends on the type of vibration (transverse or longitudinal), specimen dimensions, and specimen shape.

Similarly, the dynamic shear modulus, G_d, is obtained from the expression

$$G_d = BMn^2 \quad \text{MPa} \tag{14.10}$$

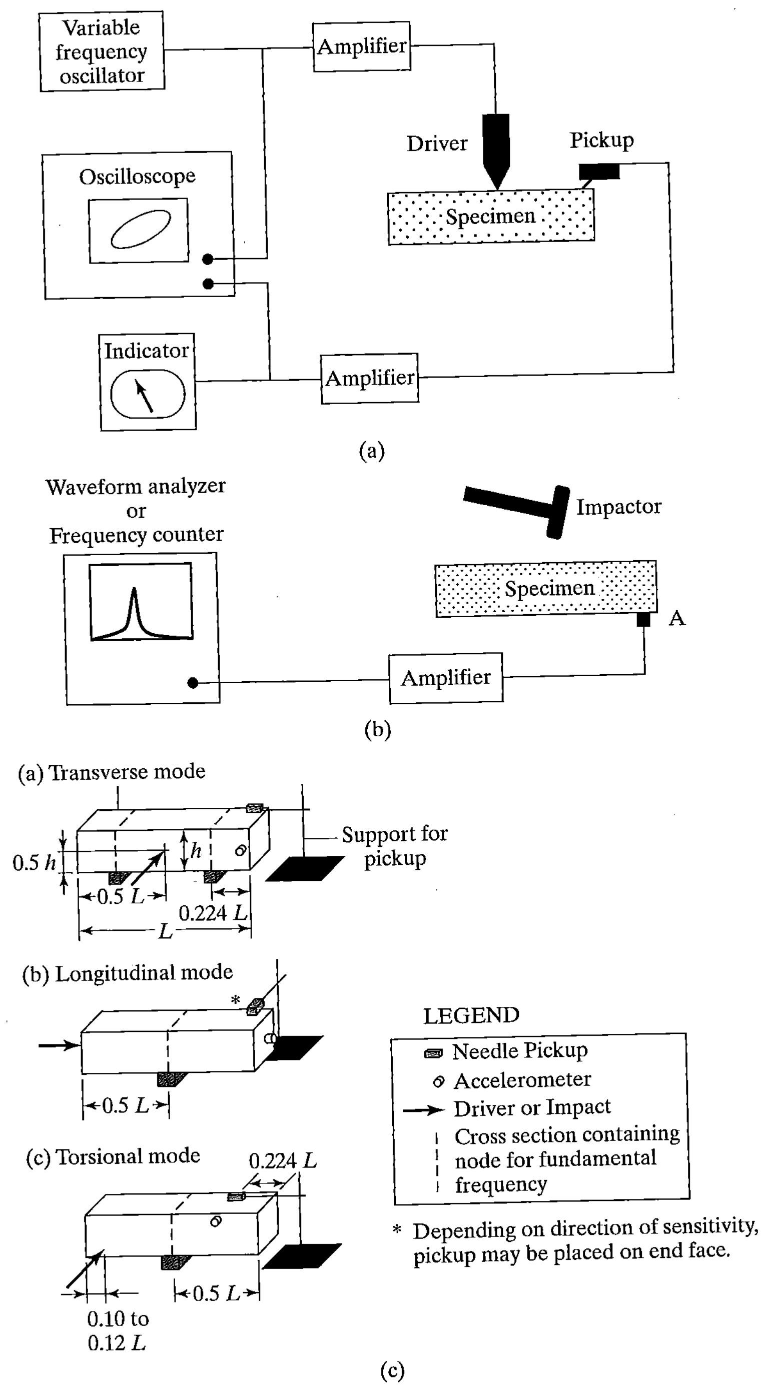


FIGURE 14.10

Schematic of apparatus for (a) forced resonance test and (b) impact resonance test; (c) locations of driver or impact and pickup or accelerometer. (From ASTM C 215. Copyright ASTM INTERNATIONAL. Reprinted with permission.

where B is a constant depending on the shape and size of the specimen. ASTM C 215 gives formulas for the calculation of K and B for various specimen geometries.

Finally, the dynamic Poisson's ratio, ν_d, can be obtained from

$$\nu_d = \frac{E_d}{2G_d} - 1 \tag{14.11}$$

In general, the dynamic elastic modulus determined in this way will be considerably higher than the static modulus of elasticity, since the dynamic modulus is approximately equal to the initial tangent modulus (see Chapter 13) rather than the chord modulus defined by ASTM C 469.

Accelerated Tests

The standard compression test was first codified in 1921, when construction proceeded at a more leisurely pace than it does today. At that time, when rather coarsely ground cements with lower C_3S contents that gained strength relatively slowly were common, acceptance tests based on the 28-day strength of the concrete were not unreasonable. However, modern developments in construction procedures, such as slipforming, have accelerated the rate at which concrete is placed, and the more finely ground, higher C_3S cements used today gain strength more rapidly. In addition, we have come to realize that the strength of standard test cylinders is, in any event, not a true representation of the strength of the concrete in the structure. Therefore, a considerable amount of work has been done to enable engineers to predict the potential 28-day strength (and hence the quality) of concrete within a few hours after casting.

To do this, it is necessary in some way to accelerate the rate of curing of the concrete, generally by the application of heat. Since the strength gain of concrete is a function of the time and temperature of curing, it should be possible to correlate the strength of concrete cured at a high temperature for a short time with the "standard" 28-day strength. It should be noted that the normal 1-day strength of concrete (or even the 3-day strength) cannot be used to predict the 28-day strength, because these early strengths are very sensitive to the fineness of the cement, the curing temperature in the first few hours after casting, and the presence of admixtures. In addition to the application of heat, some of these test methods also involve the use of accelerating admixtures or the application of pressure. However, only four of the many procedures have been developed into standards, as described in ASTM C 684:

1. *Warm water method.* Concrete cylinders are made in the normal way and then are immersed in water at 35 ± 3°C (95 ± 5°F). After $23\frac{1}{2}$ h, the cylinder is demolded, capped, and tested at an age of 24 h ± 15 min.
2. *Boiling water method.* Here, the standard cylinder is first cured in a moist environment at 21 ± 6°C (70 ± 10°F) for 23 h. It is then placed in boiling water. After $3\frac{1}{2}$ h, it is removed, cooled for 1 h, capped, and tested at an age of $28\frac{1}{2}$ h ± 15 min.
3. *Autogenous method.* This method uses the heat of hydration of the specimen to accelerate the curing. The standard cylindrical specimen is placed in a heavily insulated container and is held there for 48 h. The maximum and minimum concrete temperatures during this period are also recorded (which may indicate if

anything abnormal has occurred). The specimen is then capped and tested at an age of 49 h ± 15 min.

4. *High temperature and pressure method.* This procedure involves the simultaneous application of elevated temperature (150°C or 300°F) and pressure (10.3 MPa or 1500 psi) using special containers. The total curing period is 5 h. The specimens are then tested within 15 min of the end of the curing period.

Interpretation of Accelerated Curing Results The purpose of these tests is to provide a very early indication of the potential strength of the concrete. These tests are generally used to predict the 28-day strengths, on which most design procedures are based. They have about the same variability as conventional tests. Thus, there is no reason why the results cannot be used directly for design purposes, although this would require suitable changes in the strength values now used for design. It must be emphasized that the values obtained from these four accelerated tests are not equal to each other. Neither are they equal to the standard 28-day strength, being generally lower. They are merely different numbers that may be used to evaluate the concrete quality. It should be noted also that there is really no "universal" curve that can be used to obtain 28-day strengths from any one of these accelerated methods. If accelerated tests are to be used as predictors of the 28-day strength, they must be carried out on the same materials. It has been found that the ratio of accelerated strength to 28-day strength increases as the cement content increases and as the initial mixing temperature increases.

14.5 ASSESSMENT OF CONCRETE QUALITY

Core Tests

So far, we have discussed only tests carried out on companion samples of concrete that purport to represent the concrete in the structure, even though we have seen that this relationship can be a tenuous one. However, situations arise where it is desirable to have some measure of the strength of the concrete actually in the structure. This is the case particularly when it is suspected that low cylinder strengths are due to improper specimen preparation. The appearance of cracking or other signs of distress may also warrant an investigation of concrete strength. In addition, if it is desired to use a concrete structure for a higher stress situation than the one it was originally designed for (i.e., trying to increase allowable loads on the structure), a study of both the concrete strength and the position and size of the reinforcing steel may be necessary.

The common way of measuring the strength of the concrete in the structure is to cut cores using a rotary diamond drill (ASTM C 42). These cores (which may contain some embedded steel) are then conditioned for moisture content as described in the standard, capped, and tested in the usual way. As mentioned earlier, if the l/d ratio of the cores is less than 2.0, the core strengths must be corrected by the appropriate factor. Although this seems like a perfectly straightforward way of assessing concrete quality, there are a number of problems in interpreting the strength values obtained:

1. The strengths of cores are generally lower than those of standard cylinders, because the curing of concrete on site (where it is allowed to dry out and is subject to temperature variations) is not as favorable to strength development as curing in a moist room. Also, it is possible that some damage to the concrete occurs due to vibration of the core drill. However, the design considerations are based on standard cylinders rather than on the true strength that the concrete develops in the structure, so it is far from clear what the significance of low core strength is. (Of course, there is considered to be no problem if core strengths are higher than the specified concrete strength.)
2. The ratio of core strength to cylinder strength is not constant. It decreases as the strength of the concrete increases, from a ratio of about 1.0 for cylinder strengths of 20 MPa (3000 lb/in.^{2}) to 0.7 for cylinder strengths of 60 MPa (9000 lb/in.^{2}).
3. The strength of the core will depend on its position in the structure. Generally, cores taken near the top surface of a structural element (or near the top of a lift) are weaker than those at the bottom, simply because of the effects of bleeding and of the settlement of the coarse aggregate.
4. Concrete is anisotropic, since bleeding can cause the creation of a weak cement–aggregate bond under aggregate particles (Figure 9.7, Section 9.1). As may be seen from Figure 14.11, these planes of weakness are always horizontal in the concrete as cast, because of the influence of gravity. Therefore, they will tend to be perpendicular to the applied load (Case I) for specimens cast with the axis of loading vertical, and parallel to the applied load (Case II) for specimens cast with the axis of loading horizontal. Cracks perpendicular to the applied load affect tensile

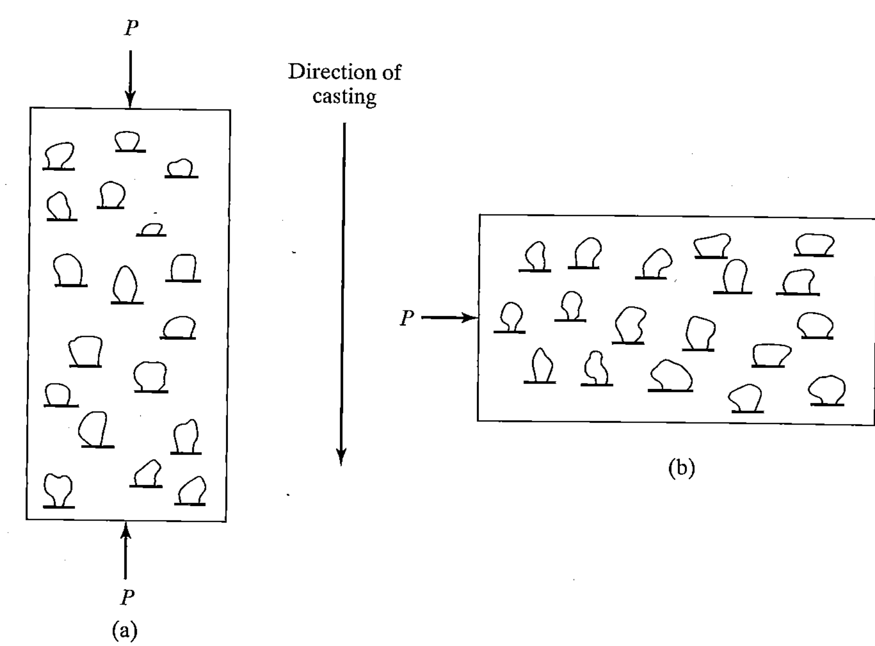


FIGURE 14.11
Planes of weakness due to bleeding: (a) axis of specimen vertical; (b) axis of specimen horizontal.

loads much more than compressive loads; the opposite is true for cracks parallel to the direction of loading. Therefore, planes of weakness due to bleeding will lead to a greater reduction in tensile strength for Case I and a greater reduction in compressive strength for Case II. As discussed in Section 13.4 (see Figure 13.26), it has been found experimentally that the strength of concrete cast with the axis of loading vertical may be as much as 10% higher in compression, with an average of 4 to 8% higher than the strength of concrete cast with the axis of loading horizontal. These values seem to be independent of the mix parameters. Therefore, when trying to evaluate the concrete strength from drilled cores, this effect should be considered when cores are drilled horizontally, as in walls or columns.

5. The core strength is affected by both the moisture content and by moisture content gradients between the surface of the specimen and the interior. The usual conditioning periods recommended in ASTM C 42 (48 hours to 7 days) are generally too short for a uniform moisture content to be established throughout the specimen. It would appear that cores dried in air for seven days are from 5–9% stronger than "as drilled" cores.

To get a proper statistical sample of concrete from the structure under consideration, many cores may have to be obtained. Thus, in addition to the problems of interpretation, cores are expensive and time-consuming, besides leaving holes in the structure that must be repaired.

Nondestructive Tests

A great deal of work has been done since the late 1940s to try to develop rapid, nondestructive tests that would provide a reproducible measure of the quality of concrete in a structure. Many such tests have been proposed, but they all still lead to difficulties in interpretation of the results. While the development of a highly reliable in situ test to monitor concrete quality remains an elusive goal, some of the tests are useful for several reasons: (1) They can be used as a quality control measure, (2) they can help determine the time for form removal, and (3) they can help in the assessment of the soundness of the concrete in existing structures (e.g., after a structure has been damaged by fire). However, it must be remembered that the tests described subsequently do *not* measure concrete strength; rather, they attempt to provide an *estimate* of the concrete strength through correlation with some other property. Unfortunately, as is usually the case in concrete testing, all of these nondestructive tests give results that are affected by a number of parameters—aggregate type and size, age, moisture content, mix proportions, and other variables. Therefore, the correlation between the measured property and strength is different for different concretes and must be determined for the particular concrete in question. The tests are useful primarily for indicating differences in concrete quality from one part of a structure to another. They may thus indicate those portions of the structure that require much closer examination, which will usually involve the drilling of some cores and possibly petrographic or other examinations.

The available nondestructive in situ tests can be classified in a number of different categories. Only a few tests have been adopted as ASTM standards, but this does not imply that the tests listed in ASTM are in any way better or more accurate than the

other tests available; it simply indicates the new tests are being developed more rapidly than the rate at which standards can be proposed, written, and adopted.

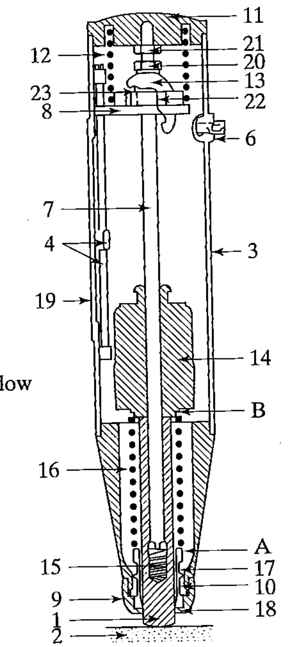
Rebound Hardness Probably the most common nondestructive test is the rebound test, using a Schmidt rebound hammer. This device (Figure 14.12a) was developed in 1948 and is universally used because of its simplicity. The test measures the *rebound* of a hardened steel hammer impacted on the concrete by a spring. Although there is no theoretical relationship, empirical correlations between rebound hardness and strength can be obtained (Figure 14.12b). This method is described in detail in ASTM C 805. The results will be affected by the following parameters:

1. *Surface finish of the concrete being tested.* Troweled surfaces give higher values than formed surfaces, and ground and unground surfaces cannot be compared.
2. *Moisture content of the concrete.* Dry concrete gives higher values than does wet concrete.
3. *Temperature.* Frozen concrete will give very high values and must be thawed before testing; the temperature of the hammer will also affect the rebound number.
4. *Rigidity of the member.* Stiffer (more rigid) members will give higher readings.
5. *Carbonation.* Carbonation of the surface can increase the hardness values by as much as 50%.
6. *Direction of impact.* The orientation of the hammer (upward, downward, horizontally, or at an angle) affects the reading since gravity affects how far the hammer mass will move.

The general view held by many users of the Schmidt rebound hammer is that it is useful in checking the uniformity of concrete and in comparing one concrete against another, but that it can only be used to obtain a rough indication of the concrete strength in absolute terms.

Penetration Resistance This type of test involves measurement of the resistance of concrete to penetration by a steel probe driven by a given amount of energy, as described in ASTM C 803. The most common device of this type is the Windsor probe (Figure 14.13). This consists of a powder-activated driving unit that "fires" a probe into the concrete; the depth of penetration (or operationally, the exposed probe length) is measured, and this can be correlated with strength. Since in this technique there is considerable penetration into the concrete, surface texture and carbonation have less effect than in the rebound test previously described. However, mix proportions and material properties are still important, and the device must be calibrated for the material in question. Harder aggregates tend to give higher apparent compressive strengths.

Pull-Out Tests Pull-out tests involve the determination of the force required to pull a steel insert out of the concrete in which it was either embedded *during casting* (Figure 14.14) or installed *after casting* (Figure 14.15). A suitable apparatus for this purpose is described in ASTM C 900. Assuming that the failure surface is a frustum, the nominal normal stress at pullout, f_n, can be calculated using the equations:



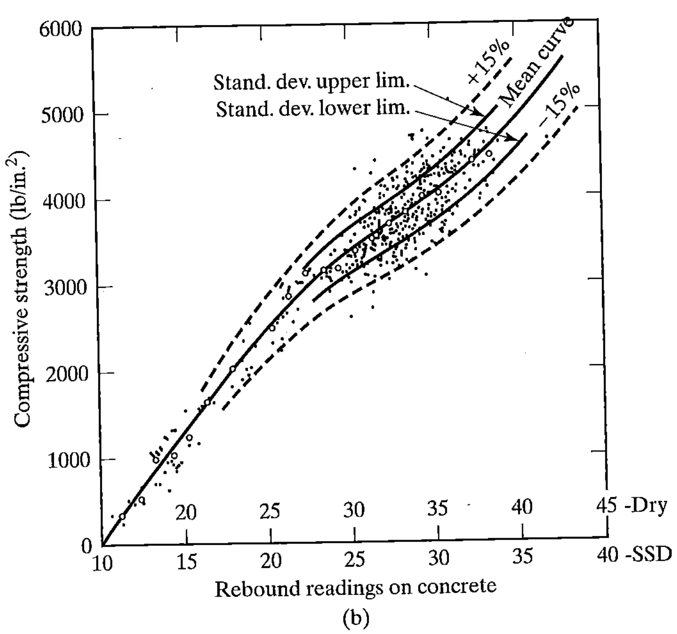


FIGURE 14.12

Schmidt rebound hammer: (a) Longitudinal section of the Type N concrete test hammer (condition on impact). (Reproduced courtesy of Proceq. S.A.); (b) calibration chart for concrete made with crushed limestone and natural sand aggregates. Five hundred standard 6 × 12-in. (150 × 300 mm) cylinders tested SSD at 28 days. Test hammer was calibrated in the horizontal position. Add five points for downward direction; deduct five points for upward direction. [From N. Zoldners, *Journal of the American Concrete Institute*, Vol. 54, No. 2, pp. 161–165 (1957).]

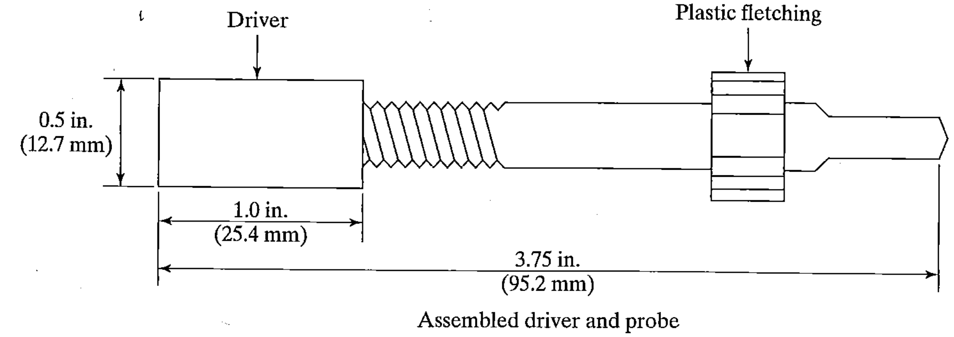


FIGURE 14.13

Windsor probe: assembled driver and probe. [From V. M. Malhotra, Department of Energy, Mines and Resources, Mines Branch Investigative Report IR-71-3 (1971). Reproduced by permission of the Minister of Supply and Services, Canada.]

$$f_n = (P/A)\sin\alpha \tag{14.12a}$$

$$\sin\alpha = (d_3 - d_2)/2s \tag{14.12b}$$

$$A = \pi S(d_3 + d_2)/2 \tag{14.12c}$$

$$S = \sqrt{h^2 + ((d_3 - d_2)/2)^2} \tag{14.12d}$$

where f_n = nominal normal stress, MPa;
P = pullout force, N;
α = ½ the frustum apex angle = $\tan^{-1}(d_3 - d_2)/2h$;
A = fracture surface area, mm²;
d_2 = diameter of pullout insert head, mm;
d_3 = inside diameter of bearing ring, mm;
h = height of conic frustum, mm;
S = slant height of the frustum, mm.

Essentially, this provides some measure of the *shear strength* of the concrete, and this can, for any particular system, be correlated with compressive strength.

The test is economical and rapid, although it does leave a hole in the concrete that must be repaired. It is probably better than those discussed previously, because a greater depth and volume of concrete are tested. The cast-in-place test, of course, must be planned in advance so that the assembly can be embedded in the concrete during casting. In contrast, the post-installed test, can inserted later and used to evaluate existing structures.

Ultrasonic Pulse Velocity This method is based on the fact that the pulse velocity, V, of compressional waves in a concrete body may be related to the elastic properties by the expression

$$V = \sqrt{\frac{E_d(1 - \nu_d)}{\rho(1 + \nu_d)(1 - 2\nu_d)}} \tag{14.13}$$

where E_d = dynamic modulus of elasticity, ν_d = dynamic Poisson's ratio, and ρ = density.

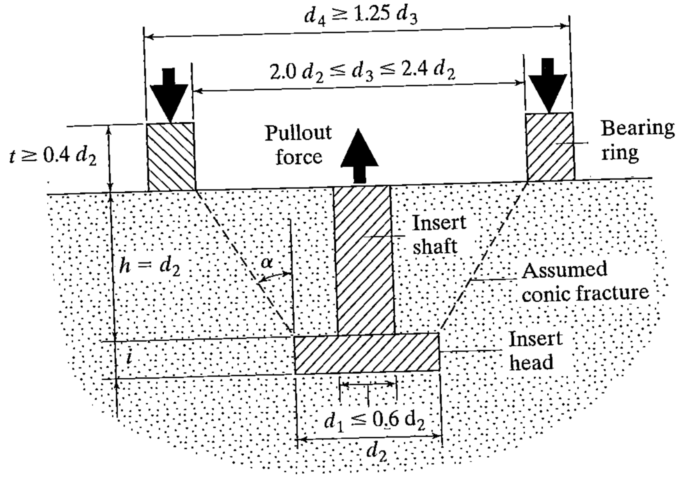


FIGURE 14.14

Schematic cross section of cast-in-place test. (From ASTM C 900. Copyright ASTM INTERNATIONAL. Reprinted with permission.)

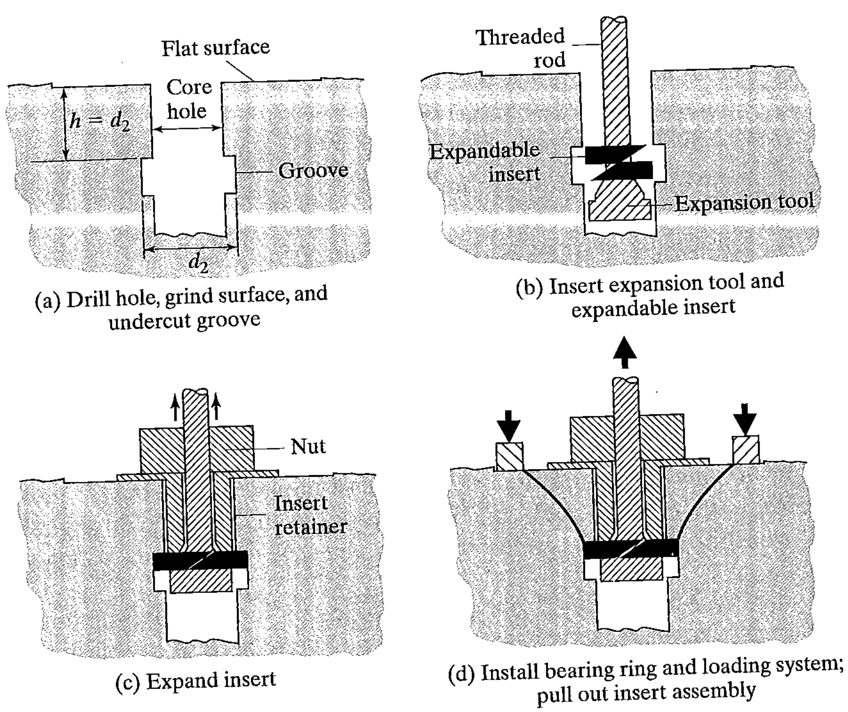


FIGURE 14.15

Schematic of procedure for post-installed pullout test. (From ASTM C 900. Copyright ASTM INTERNATIONAL. Reprinted with permission.)

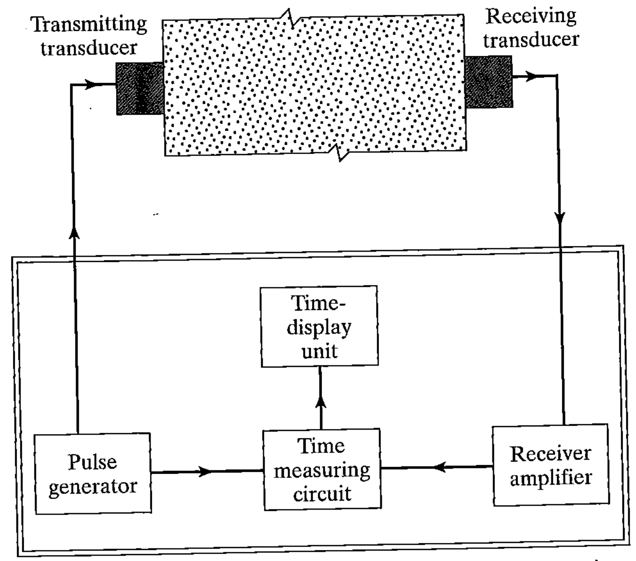
Since the pulse velocity depends only on the elastic properties of the material and not on the geometry, this is a very convenient technique for evaluating concrete quality. In essence, an apparatus such as that shown schematically in Figure 14.16a is used to determine the pulse velocity through a known thickness of concrete, using the procedures outlined in ASTM C 597. A number of commercial devices are available that meet these requirements, with an accuracy of measurement of about ±1%.

In fact, mechanical impulses applied to a material generate three types of waves: longitudinal (compressional), shear (transverse), and surface (Rayleigh). The longitudinal waves are the fastest, and they are the ones that are most useful for testing purposes. The pulse velocity may be used directly as a quality control measure, but more commonly it is correlated with strength, as shown in Figure 14.16b. However, more recent work shows this correlation to be a poor one. Pulse velocity can be measured in several configurations, as shown in Figure 14.17, although Method A seems to give the best results. The pulse velocity is affected by a number of factors:

1. *Contact surface smoothness.* If the surfaces are not reasonably smooth, they should be ground smooth; a coupling medium such as grease must also be used to ensure good contact between the transducers and the concrete.
2. *Path length.* The pulse velocity seems to depend on the path length, decreasing somewhat as the path length is increased.
3. *Temperature.* The pulse velocity is not sensitive to temperature in the range 5 to 30°C (40 to 85°F). At higher temperatures, the pulse velocity is decreased; at temperatures below freezing, it is increased.
4. *Moisture content.* Pulse velocity increases with increased moisture content.
5. *Reinforcing steel.* The presence of steel bars will tend to increase the pulse velocity.
6. *Concrete strength.* For a given compressive strength, the pulse velocity is lower for older concrete (Figure 14.18).

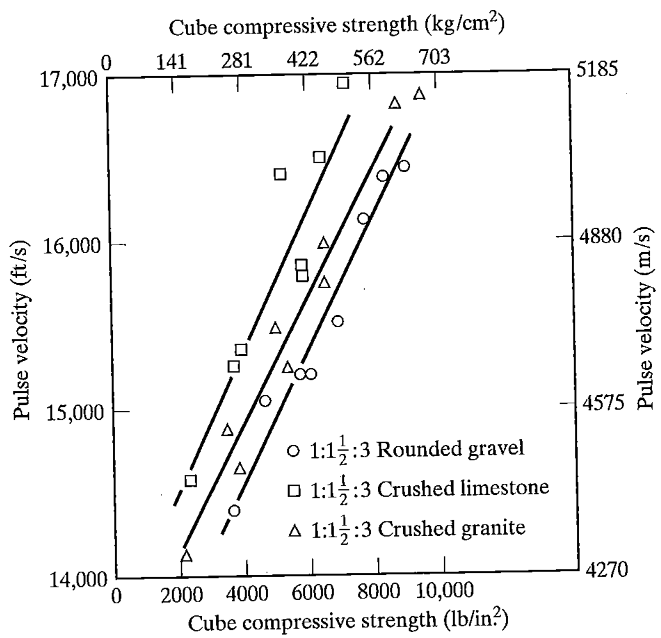
In spite of these difficulties, pulse velocity measurements are used regularly as a means of assessing concrete quality. They are used to study uniformity, for quality control, estimations of strength, setting characteristics, durability, and the extent of cracking in a member, as well as to determine the modulus of elasticity.

Impact-Echo Method The impact-echo method uses transient stress waves to detect flaws within concrete structures. Briefly, the concrete surface is mechanically impacted. This causes dilatational (P) waves, distortional (S) waves, and Rayleigh (R) waves to propagate in the concrete. Since the P and S waves are reflected from the boundaries of the member or from internal flaws, monitoring of the displacements caused by these stress waves may be used to determine both member thickness (or length) and the location of large flaws or discontinuities within the structure. This technique is still under development; it is described in detail in ASTM C 1383.



Note — It is advantageous to incorporate the pulse generator, time-measuring circuit, receiver amplifier, and time display into one unit.

(a)



(b)

FIGURE 14.16

(a) Schematic diagram of pulse velocity testing circuit. (From ASTM C 597. Copyright ASTM INTERNATIONAL. Reprinted with permission.)
(b) Correlation of pulse velocity with compressive strength. [From R. Jones, *Non-destructive Testing of Concrete*, Cambridge University Press, Cambridge, England, (1962).]

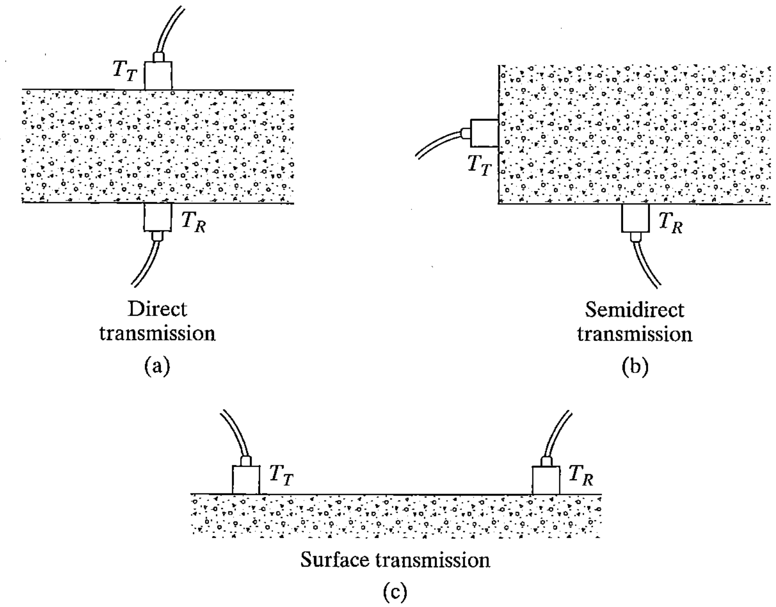


FIGURE 14.17

Methods of measuring pulse velocity through concrete: (a) direct transmission method; (b) semidirect transmission method; (c) surface transmission method.

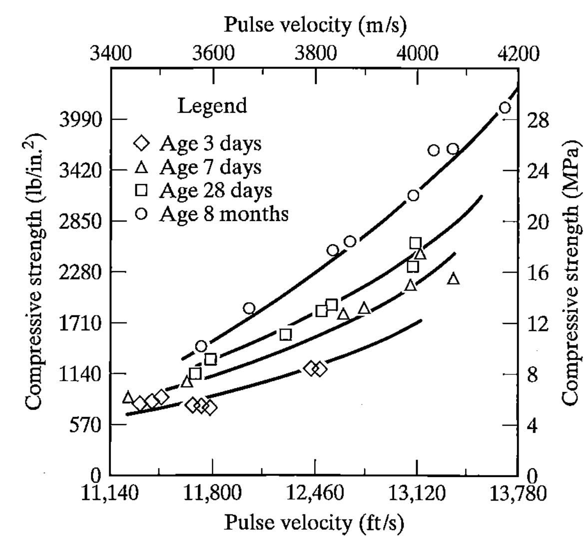


FIGURE 14.18

Influence of age of concrete on the correlation between pulse velocity and strength. [From I. Facaoaru, *Proceedings, Symposium on Nondestructive Testing of Concrete and Timber*, Institution of Civil Engineers, London, pp. 23–33 (June 1969). Reproduced by permission of the Institution of Civil Engineers.]

Other Techniques A number of other techniques have been studied that can provide some information about the concrete. These will be mentioned very briefly, as their detailed description is beyond the scope of this book. Both X rays and gamma rays will penetrate concrete to some degree. X rays are used primarily as a laboratory technique to examine the internal structure (i.e., cracking, aggregate distribution, etc.) of concrete. Gamma radiography has been used to locate reinforcing bars and to measure the density and thickness of concrete. Neutron backscattering has been used to measure moisture content, and neutron activation analysis has been used to determine cement content. A number of magnetic devices are available that will measure the depth of cover of reinforcement as well as the position of the reinforcing bars.

Analysis of Hardened Concrete

Occasionally, the question arises as to whether the concrete was cast at the specified mix composition. It is then desirable to have some method of determining the composition of the hardened concrete. This is a very difficult proposition because of the variability of the materials involved. In principle, it would be desirable to be able to determine (1) the original cement content and type of cement, (2) the *w/c* ratio, (3) the aggregate content and grading, (4) the amount and type of admixtures used, and (5) the nature of the air-void system.

In Chapter 8, it was noted that it was possible to determine the nature of the air-void system by examining a polished section of the concrete under a microscope, using a "point-count" method to determine the volume and spacing of air voids in hardened concrete (ASTM C 457). This method can be extended (ASTM C 856) to give a good indication also of the relative amounts of coarse and fine aggregate and the amount of cementitious matrix. The volumetric composition of the material is obtained from an observation of the frequency with which each component coincides with a regular array of points on a plane section through the material. Thus, it is possible to make at least some estimates of (3), part of (4), and (5) in the previous paragraph.

Unfortunately, there is as yet no reliable method for determining either the type or amount of chemical admixtures that may have been used. Thus, the determination of the original mix composition is both an expensive and a difficult problem, made worse by the fact that in the interval between casting and testing, the concrete may have been subject to leaching by chemical attack and to carbonation.

In general, no one method is sufficient to assess the quality of concrete in an existing structure. Rather, depending on the particular circumstances, a number of different tests might be used, such as hardness tests, core tests, and petrographic examination of the concrete. The object is to get the required information as economically as possible, with due consideration to the safety of the structure and the consequences of incorrect analysis. This area of concrete technology is still fraught with uncertainty and should not be lightly undertaken by inexperienced investigators.

Maturity Concept

The hydration of cement is greatly affected by both the time and the temperature of hydration, so the gain of strength of concrete is also largely controlled by these two factors. Thus, it was only natural that a considerable amount of research would be carried out to determine how the strength of concrete could be expressed as some function of

the time and temperature of curing. This information could then be used to estimate the strength of concrete without the necessity of carrying out physical tests. Out of these studies came the concept of concrete "maturity," which may be defined either as some function of the product of the curing time t and the concrete curing temperature T [i.e., maturity equals $f(T \times t)$,] or as an equivalent age for concrete cured at some specified temperature. The assumption then is that for any particular mix, concretes of the same maturity will have about the same strength, regardless of the combination of time and temperature leading to the maturity. Procedures for calculating maturity and estimating concrete strength are described in ASTM C 1074 and C 918, respectively.

To apply the first concept of maturity, it is necessary to establish some datum point from which to measure temperature (i.e., the temperature below which the concrete will show *no* increase in strength with time). Different values for this datum temperature have been found by different investigators: These values have ranged from 5 to −20°C (41 to −4°F). Values of 0°C (32°F) and −10°C (14°F) are commonly used. A number of maturity functions have been proposed over the years.

A function that shows good correlation with the strength of concrete is the *Nurse-Saul expression*,

$$M(t) = \Sigma(T_a - T_0)\Delta t \tag{14.14}$$

where $M(t)$ is the maturity at age t, Δt is a time interval in days or hours, T_a is the average concrete temperature during each time interval in °C or °F, and T_0 is the datum temperature in °C or °F. In some cases, this provides a very good correlation between maturity and compressive strength, as shown in Figure 14.19, as well as between maturity and elastic modulus.

The second approach involves expressing maturity as an equivalent age t_e at a specified temperature T_s in days or hours. Based on the *Arrhenius equation*, which relates the rate of chemical reaction with temperature, Freiesleben Hansen and Pederson proposed the expression

$$t_e = \Sigma e^{-Q\left(\frac{1}{T_a}-\frac{1}{T_s}\right)}\Delta t \tag{14.15}$$

where Q is the activation energy divided by the gas constant, in °K; T_a is the average temperature in °K of the concrete during the curing interval, Δt; and T_s is the specified temperature in °K (°K = °C + 273). The value of the activation energy depends on the cement admixtures and *w/cm* ratio. For Type I cement, Q is approximately 5000°K based on an activation energy between 40,000 and 45,000 J/mol (the gas constant is 8.31 J/°K-mol). The specified temperature T_s is typically taken as 20°C (293°K) or 23°C (296°K).

Either of the maturity concepts [Eq. (14.14) or (14.15)] may be combined with a maturity–strength function, such as that proposed by Plowman:

$$S_m = a + b \log m \tag{14.16}$$

where S_m is the compressive strength at maturity m [expressed as either $M(t)$ from Eq. (14.14) or as t_e from Eq. (14.15)].

When m is expressed as an equivalent age t_e, a represents the strength at one day at the specified temperature T_s. When m is expressed as $M(t)$, a has no physical meaning. a and b depend on the mix proportions and materials used. For a given set of

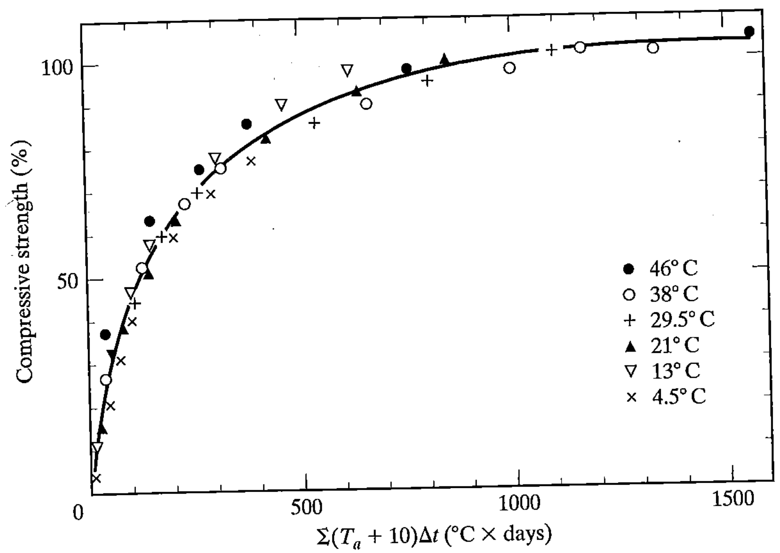


FIGURE 14.19

Percentage compressive strength as a function of the product of age and temperature (100% = 28 days at 23°C). [From S. G. Bergstrom, *Magazine of Concrete Research*, Vol. 5, No. 14, pp. 61–66 (1953). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

materials, a is sensitive to the *w/cm* ratio, while b is not. The constants a and b can initially be established in the laboratory for the materials used on the job. Then a can be recalculated based on early age cylinder strengths obtained on the job. Thus, for concrete strengths measured on the job, the strength at any later time S_M can be calculated as

$$S_M = S_m + b(\log M - \log m) \tag{14.17}$$

where M is the maturity at the later age and S_M is the strength at maturity m.

The temperature T_a in Eqs. (14.14) and (14.15) can be monitored using thermocouples or thermistors that are placed within the concrete. Automatic monitoring systems, known as maturity meters, will record temperature versus time and/or the equivalent age t_e. Experimental procedures for establishing the datum values, T_0 and Q, are described in ASTM C 1074.

Plowman stated that the combination of Eqs. (14.14) and (14.16) was valid for times up to at least one year and curing temperatures below 38°C (100°F). In addition, he provided values for the constants a and b for different strength classes of concrete. Based on Eq. (14.16) or (14.17), a graph of strength versus $\log_{10}$ (maturity) would be linear, as shown in Figure 14.20.

Unfortunately, there are a number of limitations to the use of maturity in predicting concrete strengths.

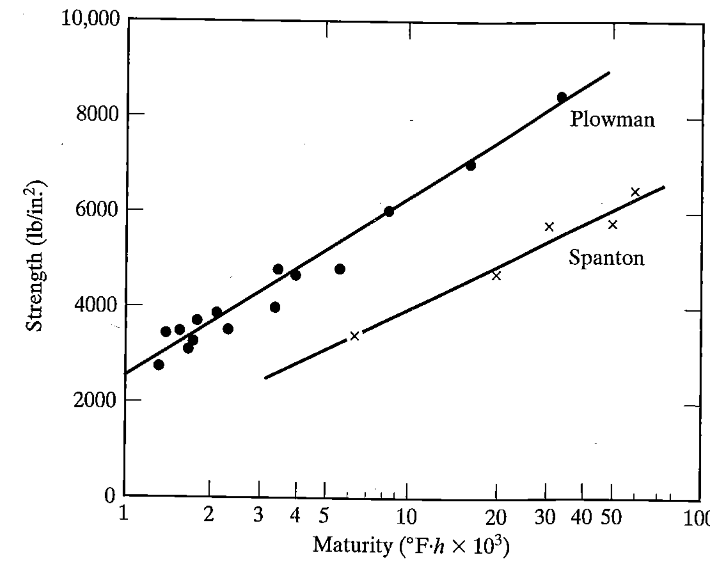


FIGURE 14.20

Strength versus log maturity. [From J. M. Plowman, *Magazine of Concrete Research*, Vol. 8, No. 2, pp. 13–22 (1956). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

1. The maturity functions that are commonly used do not take into consideration the effect of humidity conditions during curing. However, it is clear from Figure 12.1 (Section 12.1) that this is a major consideration that cannot be ignored.
2. The maturity concept is not very useful if there are large temperature variations during the curing period, since a low initial curing temperature followed by normal curing will lead to higher concrete strengths than if the concrete had been cured at a normal temperature for the total time, while a high initial curing temperature followed by normal curing will have a detrimental effect. This is indicated in Figure 12.2c (Section 12.1), where the temperatures shown on the various curves refer to the initial concrete temperatures for concrete that is subsequently cured at 21°C (70°F).
3. Although there seems to be some correlation between maturity and strength for the accelerated strength tests described earlier in this chapter, the relationship breaks down when the accelerated cure specimens are subsequently cooled and moist cured.

For the best accuracy, the datum temperature or activation energy and the constants in Eq. (14.16) must be measured by tests (ASTM C 918 and C 1074) since they are affected by the chemical composition of the cement and the admixtures and the fineness of the cement, which affect the rate of hydration. The *w/cm* ratio will also influence the results.

Some investigations have found that maturity can be used to describe concrete strengths over a wide range of times and temperatures and concrete mixes. Others have found that the maturity relationships work only over very narrow ranges of time or temperature, or not at all. There seems to be a general agreement that the equivalent age concept represented by Eq. (14.15) provides a somewhat better representation than the time–temperature relationship represented in Eq. (14.14). Under any circumstances, the process of strength gain in concrete is very complex and cannot be generally modeled by the simple functions described earlier. In particular, these functions should in no way be considered as physical "laws."

The maturity concept is not widely used, probably because of the uncertainties described, but concrete technologists should be aware of the method of estimating concrete strength with all of its inherent limitations. Nevertheless, in spite of its limitations, the maturity concept may be useful when trying to establish "after the fact" the strength of concrete in a structure at a previous time. This may be done by measuring core strengths at some later time and then using one of the maturity functions to estimate the strength at an earlier time. Also, the maturity concept can be used to estimate the appropriate time for form removal when concreting at lower-than-normal temperatures.

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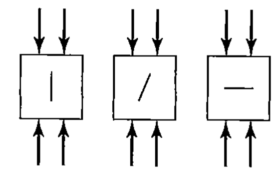
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PROBLEMS

14.1. Why are tests carried out on hardened concrete?
14.2. What is the purpose of having detailed specifications in standard tests?
14.3. Compare cylinder and cube strengths for compressive strength.
14.4. Why does increasing the rate of loading increase the apparent strength?
14.5. Why is it necessary to cap concrete cylinders?
14.6. (a) How would cracks oriented as in (1), (2), or (3) affect the measured compressive strength?
(b) How would these cracks affect the tensile strength?



(1) (2) (3)

14.7. Why is the l/d ratio a sensitive parameter in the compressive strength test?
14.8. Why does the moisture content of a concrete specimen affect its strength?
14.9. Compare the methods of measuring tensile strength by (a) direct tension; (b) splitting tension; (c) flexure.
14.10. Why are accelerated tests of concrete strength useful?
14.11. What are the problems involved in interpreting the compressive strength of cores taken from a concrete structure?
14.12. Why are nondestructive tests used?
14.13. Can nondestructive tests give an absolute measure of strength? Discuss.
14.14. What test methods can be used to detect internal cracking in concrete?

CHAPTER 15

Quality Control

By now, we have discussed the many factors that are involved in the production of high-quality concrete: materials, proportioning, handling and placing, curing, and testing. It should, therefore, come as no surprise that concrete, in common with other engineering materials, is inherently a *variable* material. That is, tests on nominally identical samples of concrete will show some variation in mechanical properties between samples. Clearly, this variability in properties must be considered when writing concrete specifications.

In general, the factors that contribute to this variability may be grouped as follows:

1. *Materials.* This includes variability in the cement itself; in the grading, moisture content, mineral composition, physical properties, and particle shape of the aggregates; and in the admixtures used.
2. *Production.* This involves the type of batching plant and equipment, the method of transporting the concrete to the site, and the procedures and workmanship used to produce and place the concrete.
3. *Testing.* This includes the sampling procedures, the making and curing of test specimens, and the test procedures used.

It is, of course, very difficult to assess the relative importance of these three groups of factors; in any event, their importance will vary for different regions and different construction projects. Since the variability in concrete quality is some function of the variabilities of each of these three factors, no one of these can be ignored in concrete production.

The term "quality assurance" is frequently used in the construction industry. Quality assurance refers collectively to all of the steps taken to ensure adequate confidence that the concrete will perform satisfactorily in service. Quality control applies to each action used to measure the properties of the concrete, or its components, and to control them within the established specifications.

15.1 MEASUREMENT OF VARIABILITY

Before discussing quality control, however, it is necessary to define the concept of variability more precisely. It has been found that the *distribution* of concrete strengths can best be approximated by the *normal (Gaussian)* distribution. Such a distribution is completely defined by two parameters: the mean, μ, and the standard deviation, s. The equation of the normal distribution curve is

$$y = \frac{1}{s\sqrt{2\pi}}\exp\left[\frac{-(x-\mu)^2}{2s^2}\right] \tag{15.1}$$

The mean is simply the arithmetic mean of all the values. If we represent the values of strength by x, then

$$\mu = \frac{\Sigma x}{n} \tag{15.2}$$

The standard deviation or the *root-mean-square* deviation is a measure of the *dispersion*, or variability, of the values. It can be calculated by

$$s = \sqrt{\frac{\Sigma(x-\mu)^2}{n-1}} \tag{15.3}$$

The value of s is more easily obtained with a calculator if Eq. (15.3) is rewritten in the form

$$s = \sqrt{\frac{\Sigma x^2 - (\Sigma x)^2/n}{n-1}} \tag{15.4}$$

The *variance* is defined as s^2, and the *coefficient of variation, V*, as

$$V = \frac{s}{\mu} \times 100\% \tag{15.5}$$

It is possible to "normalize" Eq. (15.1) (i.e., make it free from physical units) by making the substitution

$$z = \frac{x-\mu}{s} \tag{15.6}$$

The value z simply represents the number of standard deviations that x is away from the mean. Equation (15.1) can then be written

$$y = \frac{1}{s\sqrt{2\pi}}e^{-z^2/2} \tag{15.7}$$

When the function described by Eq. (15.1) or (15.7) is graphed, it gives the familiar bell-shaped curve shown in Figure 15.1 in terms of both x and z units. This curve has the following properties, the proofs of which can be found in any elementary text on probability theory:

1. The maximum ordinate is at $z = 0$ (or $x = \mu$).
2. The curve is symmetrical about $z = 0$.

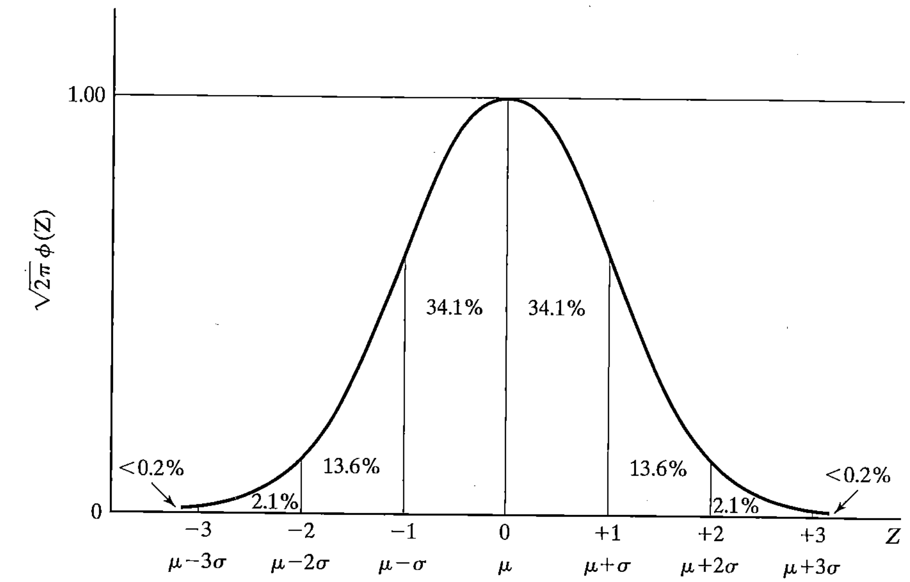


FIGURE 15.1
Areas under the normal curve.

3. Points of inflection exist at ± 1 standard deviation from the mean, that is, at $z = \pm 1$ (or $x = \mu \pm s$).
4. The total area under the normalized curve is 1.
5. The area under the curve between any two points is the probability of occurrence of a value in that interval. That is, the probability of a value lying within one standard deviation of the mean is 68.27%; within two standard deviations from the mean, 95.45%; within three standard deviations from the mean, 99.73%.
6. If we know the standard deviations $s_1, s_2, \ldots, s_n$ of all the components that contribute to the total standard deviation, the total standard deviation is given by

$$s^2 = s_1^2 + s_2^2 + \cdots + s_n^2 \tag{15.8}$$

It may be seen, of course, that such a curve cannot be a perfect representation of the variability in concrete properties, since it extends to ± infinity. However, if we assume that the only area of interest is the region within three standard deviations from the mean, which contains 99.73% of all expected values, this curve is a very useful means of characterizing concrete properties.

Applications to Concrete

Now, assuming that the variation in concrete properties is to be described by the normal distribution, let us see how these concepts are applied in practice. In the discussion that follows, reference will be to the compressive strength, because it is here that the use of the variability concept is most common. However, it must be remembered that

this concept is a general one and may be applied equally to any other quantifiable concrete properties, such as w/c ratio, flexural strength, and elastic modulus.

Once we accept the concept that there is a statistical distribution of concrete strengths, several important implications emerge:

1. We cannot design concrete structures on the basis of mean strength (or average strength) as defined by Eq. (15.2). If we did, this would mean that about one-half of the concrete placed would have strengths that fall *below* the design value, which would be unacceptable. On the other hand, we cannot insist that *all* concrete strengths be above the design value; since concrete strengths are approximately normally distributed, this is an impossibility. Therefore, we must arbitrarily decide what constitutes an acceptable percentage of specimens falling below the "minimum" design values. Using this percentage, and knowing (or assuming) the standard deviation in strength that can be expected, we can then determine the required mean strength for which to design the concrete mix.
2. When carrying out tests on concrete, we are trying to evaluate the distribution in strength of all of the concrete in the structure, based upon a limited sample size. Clearly, enough test data must be collected so that the tests are truly representative of the concrete in the structure. However, because of the rather tenuous relationship between the strengths of test specimens and the quality of concrete in the structure (Chapter 14), we can at best only *estimate* the strength of the concrete in the structure.
3. Because variations in concrete strengths are due not only to mix variations, but also to sampling variations, there are two risks that must be balanced: the "producer's risk" that satisfactory concrete will be rejected and the "consumer's risk" that bad concrete will be accepted. This consumer's risk can be large indeed if insufficient testing is carried out.
4. There must be some plan of action that can be followed if the concrete is considered not to have complied with the specifications.

It should be remembered, however, that the acceptance of the fact that a certain proportion of concrete test cylinders will be below the design strength of the concrete is much less "risky" than might appear at first glance, for several reasons:

1. Concrete batches tend to get intermixed as the concrete is placed in the forms, and this tends to "average out" the strengths.
2. As we have seen, most acceptance tests are based on 28-day strengths; however, concrete continues to gain strength beyond this time, and this will tend to compensate for low strengths by the time the concrete is actually in service.
3. In part because of the steel reinforcement of concrete, there can be a considerable redistribution of stresses in a structure.
4. Perhaps most important, many years of experience have shown that structures built in accordance with these concepts behave satisfactorily.

15.1.2 ACI Approach to Variability

ACI approaches the variability of concrete by establishing both a required average strength and acceptance criteria for individual tests.

Required Average Strength The approach of ACI Committee 214, Evaluation of Results of Tests Used to Determine the Strength of Concrete, to the variability inherent in concrete strength is to require an average concrete strength sufficiently in excess of the specified design strength so that only an allowable proportion of low strengths will occur. The required average strength is computed from the expression

$$f'_{cr} = f'_c + ts \tag{15.9}$$

where f'_{cr} is the required average strength, f'_c is the specified design strength, t is a constant depending on the proportion of tests allowed below f'_c (Table 15.1), and s is the predicted value of the standard deviation. The reliability of this expression depends on the number of samples used to establish s; preferably at least 30 samples should be used. It is also possible to estimate the desired average strength from Figure 15.2.

It should be noted that this approach provides a great deal of flexibility. Different classes of concrete could be permitted different probabilities of low-strength concrete, depending on the type of structure and on the economic and other consequences of failure. It is the opinion of Committee 214 that reasonable control of structural concrete would be obtained if the probability of a test below f'_c is no greater than 1 in 10.

However, the ACI Building Code Requirements for Structural Concrete (ACI 318) is somewhat more restrictive. Its requirements for setting the required average strength are twofold:

1. The probable frequency of the *average* of three consecutive tests below f'_c will not exceed 1 in 100. This can be written in terms of s as

TABLE 15.1 Values of t Used in Eq. (15.9)[a]

Percentages of Tests Falling within the Limits $\mu \pm ts$	*Chances of Falling below Lower Limit*	t
40	3 in 10	0.52
50	2.5 in 10	0.67
60	2 in 10	0.84
68.27	1 in 6.3	1.00
70	1.5 in 10	1.04
80	1 in 10	1.28
90	1 in 20	1.65
95	1 in 40	1.96
95.45	1 in 44	2.00
98	1 in 100	2.33
99	1 in 200	2.58
99.73	1 in 741	3.00

[a]From ACI 214. Reproduced with permission.

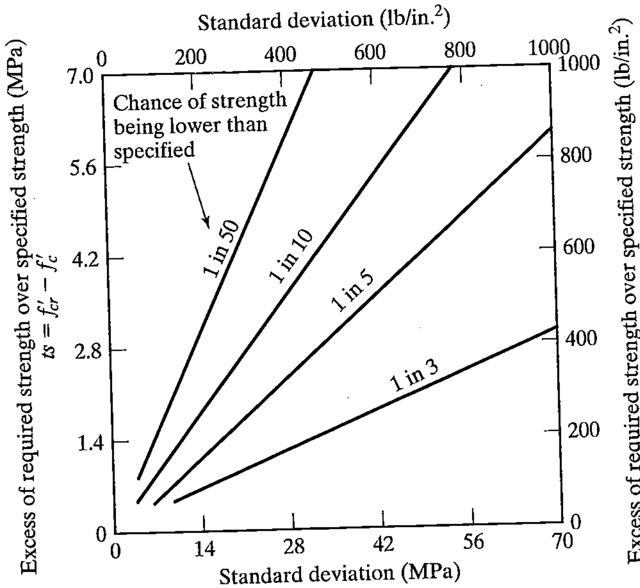


FIGURE 15.2

Excess of required average strength f'_{cr} to specified strength f'_c for various standard deviations and chances of falling below specified strength. [From *Recommended Practice for Evaluation of Strength Test Results of Concrete*, ACI 214-77 (reapproved 1997), American Concrete Institute, Farmington Hills, MI (1997).]

$$f'_{cr} = f'_c + \frac{2.33s}{\sqrt{3}} = f'_c + 1.34s \qquad (15.10)$$

2. a. For $f'_c \leq 35$ MPa (5000 psi), the probable frequency of tests more than 3.5 MPa (500 psi) below f'_c should not exceed 1 in 100. This can be written

$$f'_{cr} = f'_c + 2.33s - 3.5 \quad \text{(MPa)} \qquad (15.11a)$$

$$f'_{cr} = f'_c + 2.33s - 500 \quad \text{(lb/in.}^2\text{)} \qquad (15.11b)$$

b. For $f'_c > 35$ MPa (5000 lb/in.^{2}), the probable frequency of tests below $0.90 f'_c$ should not exceed 1 in 100. This can be written

$$f'_{cr} = 0.90 f'_c + 2.33s \qquad (15.12)$$

It should be clear that, whatever the exact criteria adopted, there is a considerable economic benefit in achieving the tightest quality control (i.e., the lowest standard deviation) possible. Figure 15.3 shows the normal distribution curves for concrete, with different standard deviations, showing the average strength f'_{cr} based on a probability of 1 in 10 that a test will fall below $f'_c = 21$ MPa (3000 lb/in.^{2}). It can be seen that for $s = 2.1$ MPa (300 lb/in.^{2}), $f'_{cr} = 23.7$ MPa (3440 lb/in.^{2}), while for 4.2 MPa (610 lb/in.^{2}), $f'_{cr} = 27.8$ MPa (4030 lb/in.^{2}). Thus, as s increases, the average design strength must be increased. Since this is done principally by adding cement, it can be quite expensive to do. The price of cement continues to rise: At the present cost of cement, it might require about \$8.50/m^3 (\$6.50/yd^3) extra to design for $s = 4.2$ MPa rather than $s = 2.1$ MPa. As energy costs continue to rise, so will the cost of cement; poor quality control will become more and more expensive. Apart from producing generally inferior concrete

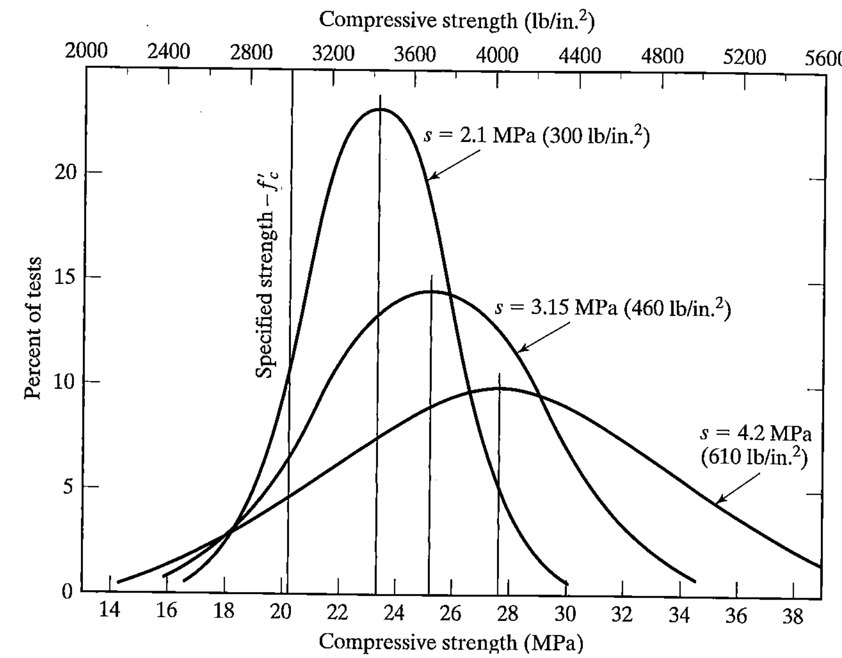


FIGURE 15.3

Normal frequency curves for standard deviations of 2.1 MPa, 3.15 MPa, and 4.2 MPa. [From *Recommended Practice for Evaluation of Strength Test Results of Concrete*, ACI 214-77 (Reapproved 1997), American Concrete Institute, Farmington Hills, MI, (1997).]

due to the implied lack of quality control, this will also lead to a "waste" of strength. On the other hand, these costs must be balanced against the costs of providing better quality control.

When there are insufficient data to use Eqs. (15.10) or (15.11) with statistical certainty (i.e., when there are less than 30 consecutive test results available), then ACI 318 requires the value of s to be modified as shown in Table 15.2. For less than 15 tests, the requirements given in Table 15.3 must be used.

Clearly, different types of construction and equipment will lead to different levels of variability. It is very difficult to decide on an acceptable level. However, based on large numbers of strength tests, Table 15.4 gives the commonly accepted standards of quality control.

Acceptance Criteria In addition to establishing criteria for f'_{cr} to ensure a low probability that the compressive strength will be below f'_c, ACI 318 provides two criteria for the *acceptance* of concrete. These criteria are based on the same reasoning used to select f'_{cr}.

For laboratory-cured specimens of job concrete:

1. Every arithmetic average of any three consecutive strength tests must equal or exceed f'_c; and

TABLE 15.2 Modification Factor for Standard Deviation when Fewer than 30 Tests Are Available[a]

No. of Tests[b]	*Modification Factor for Standard Deviation*
Less than 15	Use Table 15.3
15	1.16
20	1.08
25	1.03
30 or more	1.00

[a]From ACI 318. Reproduced with permission.
[b]Interpolate for intermediate values.

TABLE 15.3 Required Average Compressive Strength when There Are Insufficient Data Available to Establish the Value of the Standard Deviation, s[a]

Specified compressive strength f'_c, MPa (lb/in.^{2})	*Required average compressive strength f'_{cr}, MPa (lb/in.^{2})*
Less than 20 (3000)	f'_c + 7.0 (1000)
20 to 35 (3000 to 5000)	f'_c + 8.5 (1200)
Over 35 (5000)	1.1 f'_c + 5.0 (700)

[a]Adapted from ACI 318. Reproduced with permission.

2. No individual test (average of two cylinders) may fall below f'_c by more than 3.5 MPa (500 psi) when f'_c is $\leq$35 MPa (5000 psi) or by more than $0.10 f'_c$ if f'_c is >35 MPa (5000 psi).

Failure to satisfy either of these criteria requires changes in the mix proportions and construction procedures to increase concrete strength. Failure to meet the second criterion requires an investigation of the strength of the concrete *in the structure* and may require an evaluation of the strength of the structure itself (Section 15.3).

Number of Samples To carry out the sort of statistical analysis just described, a sufficient number of samples must be obtained. In addition, sampling should also provide a continuing check on the concrete being provided. A very common requirement is that adopted by the ACI, which requires that samples be taken at least once a day and at least once for every 110 m^3 (150 yd^3) of concrete or for each 450 m^2 (5000 ft^2) of surface area placed. This procedure should be followed for each type of concrete being used, with the provision that at least five samples should be obtained for every type of concrete. If the total quantity of concrete of a given type is less than 40 m^3 (50 yd^3), tests may be waived by the statutory building official. (The procedures for sampling fresh concrete are described in Chapter 9.)

To generate random samples, as required by the statistical considerations described, a sampling plan should be established in advance of the construction project. Samples should be taken from *predetermined* batches in accordance with the sampling

TABLE 15.4 Standards of Concrete Control[a]

Overall Variation					
	Standard Deviation for Different Control Standards, MPa (lb/in.^{2})				
Class of Operation	*Excellent*	*Very Good*	*Good*	*Fair*	*Poor*
General construction testing	Below 2.8 (400)	2.8–3.5 (400–500)	3.5–4.2 (500–600)	4.2–4.9 (600–700)	Above 4.9 (700)
Laboratory trial batches	Below 1.4 (200)	1.4–1.8 (200–250)	1.8–2.1 (250–300)	2.1–2.5 (300–350)	Above 2.5 (350)
Within-Test Variation					
	Coefficient of Variation for Different Control Standards (%)				
Class of Operation	*Excellent*	*Very Good*	*Good*	*Fair*	*Poor*
Field control testing	Below 3.0	3.0–4.0	4.0–5.0	5.0–6.0	Above 6.0
Laboratory trial batches	Below 2.0	2.0–3.0	3.0–4.0	4.0–5.0	Above 5.0

[a]From ACI 214. Reproduced with permission.

plan. The practice of testing only batches of high or low slump, or batches that appear to be "representative" to the inspector should be avoided, since this will not give an accurate representation of either concrete quality or uniformity.

15.2 QUALITY-CONTROL CHARTS

Statistical principles alone are insufficient for quality control—we must be able to apply them efficiently so that we can determine as quickly as possible both concrete quality and possible changes in quality. We may find, for instance, that the mean strength designed for is being achieved, but that the standard deviation is larger than expected. This would require an increase in the mean strength, by adjusting the mix. On the other hand, a lower than expected standard deviation might permit a reduction in mean strength. Equally important, it might become apparent that *changes* in either the mean strength or standard deviation have occurred at some stage of the construction process. This might be due to changes in the raw materials (e.g., moisture content of the aggregate), the batching equipment, or a variety of other reasons. In any event, the sooner such changes are detected, the sooner we can take the appropriate remedial measures.

This implies that concrete strengths must be assessed on a continuous basis. This can most effectively be done using *quality control charts* of the type shown in Figure 15.4. Three types of charts are employed; in each case, the horizontal axis represents consecutive sample numbers:

1. Figure 15.4a is a chart of the individual strength values. The line for required average strength, f'_{cr}, is obtained using Eq. (15.9), based on the allowable number of tests below the specified strength, f'_c. This chart simply shows the number of

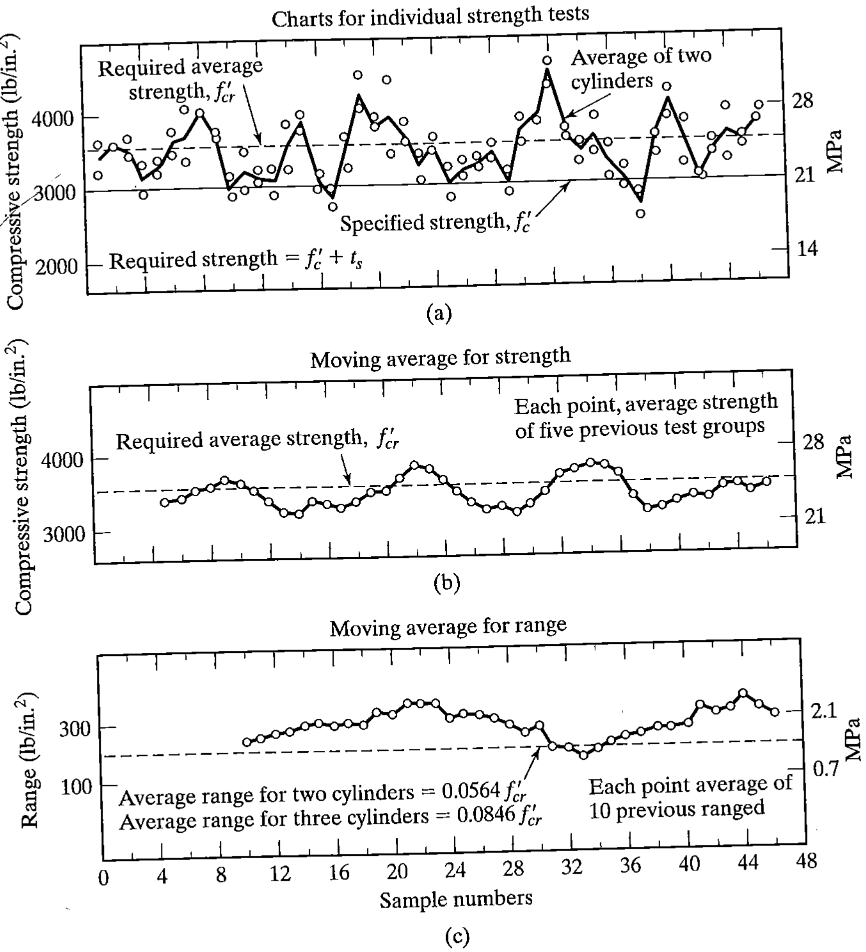


FIGURE 15.4

Quality control charts for concrete. [From *Recommended Practice for Evaluation of Strength Test Results of Concrete*, ACI 214-77 (Reapproved 1997), American Concrete Institute, Farmington Hills, MI (1997).]

"low" tests and gives some indication of the scatter. However, this chart is relatively insensitive to changes in the concrete quality.

2. Figure 15.4b is a graph of the *moving* average for strength, where each point represents the average of the previous five sets of strength tests. (Each strength test will normally consist of breaking two or three cylinders.) This chart tends to "smooth out" chance variations and can be used to indicate trends that may be due to the influence of seasonal changes, changes in procedures, and so on. The number of tests to be averaged for each point can, of course, be varied to suit the particular job requirements. For instance, since we have stated previously that

TABLE 15.5 Evaluation of Consecutive Low-Strength Test Results[a]

	Averages Less Than Indicated Require Investigation[b]		Probability of Averages Less Than f'_c(%)
	Criteria for Original Selection of f'_{cr}		
	1 Test in 10 below f'_c	1 Test in 100 Less Than f'_c − 3.5 MPa(500 lb/in²).	
Number of Consecutive Tests Averaged	For Given s	For Given s	1 Test in 10 below f'_c
1	$f'_c - 0.77s$	$f'_c - 3.5 + 0.76s$	10.0
2	$f'_c - 0.17s$	$f'_c - 3.5 + 0.88s$	3.5
3	$f'_c + 0.10s$	$f'_c - 3.5 + 1.14s$	1.3
4	$f'_c + 0.26s$	$f'_c - 3.5 + 1.30s$	0.5
5	$f'_c + 0.36s$	$f'_c - 3.5 + 1.41s$	0.2
6	$f'_c + 0.44s$	$f'_c - 3.5 + 1.49s$	0.1

[a]From ACI 214. Reproduced with permission.
[b]The probability of averages less than the levels indicated is approximately 2% if the population average equals f'_{cr} and the standard deviation or coefficient of variation is at the level assumed.

the ACI Building Code requires that the frequency of an average of three consecutive tests below f'_c will not exceed 1 in 100, a chart where each point represents the average of the three previous tests would be appropriate. Table 15.5 provides a guide for concrete strength specifications, indicating the strength level below which individual tests or averages of a series of tests should not fall. (From the statistical theory that we have assumed, we would expect that a failure to meet these specifications would occur about 1 time in 50.) A chart such as the one in Figure 15.4b would therefore have f'_c as its lower limit. A failure to meet the limitations of the chart (or Table 15.5) would indicate that the required average strength f'_{cr} is not being reached, due to lower strengths, to more variability than anticipated, or to poor testing techniques.

3. Figure 15.4c represents the moving average of the *range* (the difference between the highest and lowest values in a set), where each point represents the average of the ranges of the 10 previous sets of strength tests. The *maximum* average range for "good laboratory" control (see Table 15.5) is also shown. This chart provides a check on the adequacy of the test procedures. If poor test procedures are indicated, they must be corrected. The adequacy of the test procedures, as indicated by the range between companion cylinders comprising a strength test, can be expressed by

$$\overline{R}_M = f'_{cr} v_1 d_2 \tag{15.13}$$

where $\overline{R}_M$ is the average range, v_1 is the within-test coefficient of variation, and d_2 is a constant depending on the number of cylinders tested. Since v_1 should not exceed 5% for good control, the maximum average range should be

$$\overline{R}_M = (0.05 \times 1.128)f'_{cr} = 0.0564 f'_{cr} \text{ for two companion cylinders} \quad (15.14a)$$

and

$$\overline{R}_M = (0.05 \times 1.693)f'_{cr} = 0.08465 f'_{cr} \text{ for three companion cylinders.} \quad (15.14b)$$

15.3 FAILURE TO MEET SPECIFIED REQUIREMENTS

The application of the statistical considerations and acceptance criteria described earlier is relatively straightforward, and if the concrete is, on the basis of these considerations, judged to have complied with the specifications, then everybody (the concrete supplier, the contractor, the testing agency, and the owner) is happy. If the concrete is judged *not* to have complied with the specifications, there are rational procedures that can be applied.

The procedures may be both expensive and time-consuming, but their application will generally result in enough information to make a sensible judgment on accepting or rejecting the structure. The procedures suggested in what follows are discussed in order of increasingly severe consequences, based on the RILEM "Recommended Principles for the Control of Quality and the Judgment of Acceptability of Concrete" and shown graphically in the flowchart, Figure 15.5.

The first and simplest check is on the sampling and testing techniques, which are all too often at fault. It may be possible to determine that the test procedures were not correctly applied or that the equipment used was not calibrated correctly or was otherwise unsuitable. If this is found to be the case, and if there is no other reason to suspect the concrete quality, little or no further action will be required beyond perhaps a few tests of the in-place concrete just to make sure of its quality.

Similarly, the materials and batching plant should be investigated, to make sure that the materials used were the ones specified, that the batching equipment was producing concrete with the desired mix proportions, and that the mix design itself was adequate. If the supplier is clearly at fault, the cause of the problem is then known, and the appropriate remedial measures can be taken, at least for the subsequent concrete production. These measures might involve redesign of the mix or more careful inspection and control.

Often, however, no easily assignable reasons for the noncompliance of the concrete can be found. Even if they are found, there is still the problem of assessing the quality of the noncomplying concrete already placed. Thus, at this stage it becomes necessary to try to evaluate the quality of the concrete in the structure. Most importantly, we want to know the load-carrying capacity of the structure. It would also be desirable to find out why the concrete did not comply with the specification (if this has not already been ascertained), so that the costs of repairing the damage can be properly assigned.

For these purposes, in situ tests as described in Chapter 14, such as coring, hardness tests, and so on, can be used. Almost always, the various building codes require that some cores be taken from the structure. Sometimes that portion of the concrete represented by the low test results can be located within the structure; otherwise, problem areas can often be found by using either hardness tests or ultrasonic tests. In any event, sufficient cores should be taken to give statistically valid results. The ACI and

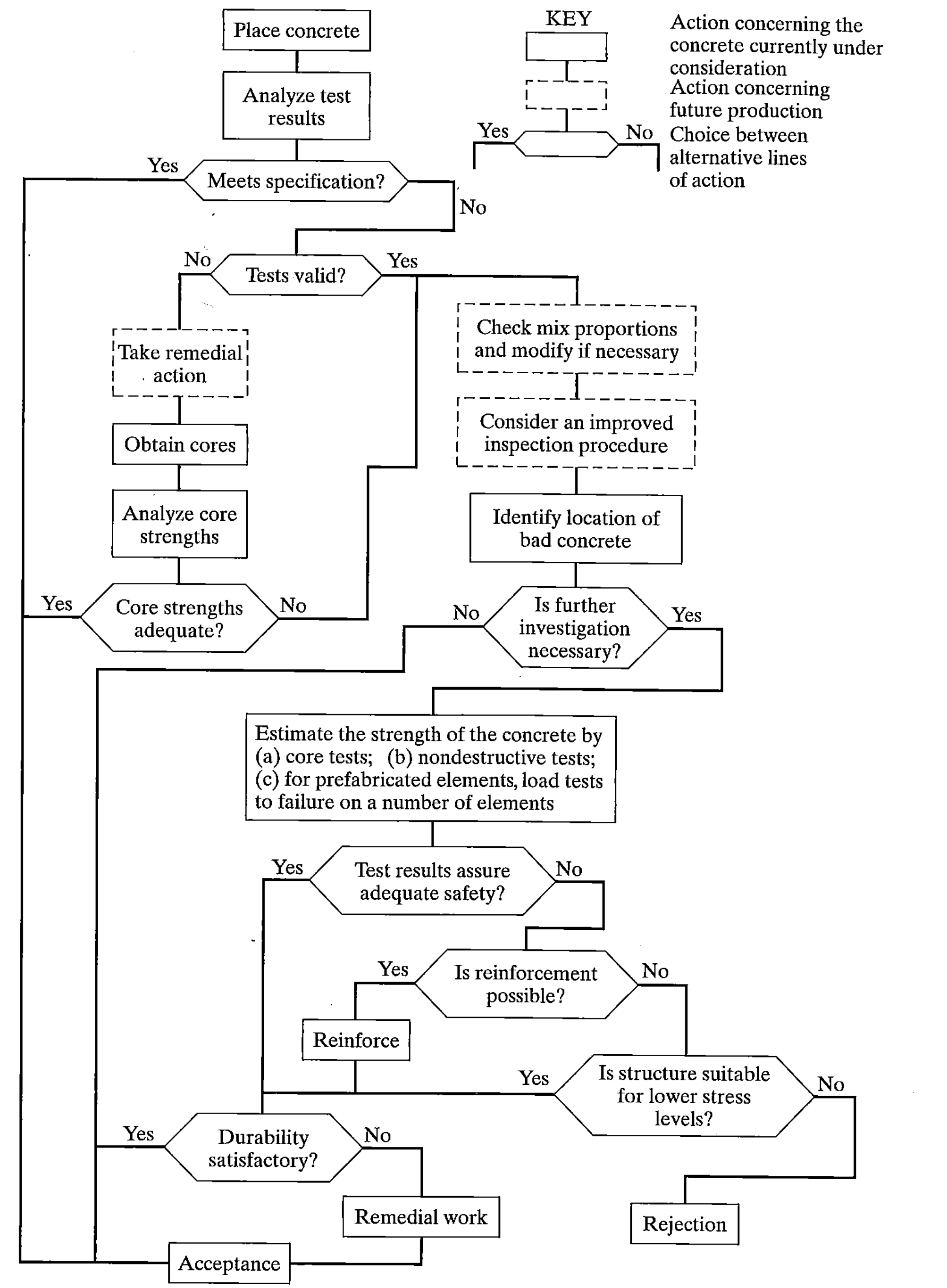


FIGURE 15.5

Flowchart for actions following judgment of compliance with specifications. [Adapted from *Materials and Structures (Paris)*, Vol. 8, No. 47, pp. 387–403 (1975).]

CSA requirement is that at least three cores should be taken for each case of a cylinder test more than 3.5 MPa (500 lb/in.^{2}) below f'_c. The concrete is considered to be adequate if the average of three cores is at least 0.85 f'_c, and no single core is less than 0.75 f'_c based on the successful performance of structures with cores meeting these criteria. There is, however, no well-established correlation between core strengths and durability (see Section 14.5, "Core Tests").

Structural Safety

At this stage, if the concrete is still judged to be inadequate, a further investigation is required to assess both the safety and the durability of the structure. This will generally involve a determination not only of the concrete strength (from core tests), but also of the location and size of the steel reinforcing. The latter information may involve removal of the concrete to expose the steel or more generally will involve the use of magnetic indicators that are available for this purpose. Once the concrete strength and the exact details of the steel reinforcement are known, the load-carrying capacity of the structure can be determined analytically, with a much higher degree of certainty than was possible in the original design. This analysis can then be used to assess the structure; it may be suitable for the intended purpose, or it may be possible to reclassify the structure for a lower level of live loads. If neither of these alternatives ensures the safety of the structure, it must be rejected, unless satisfactory repairs and strengthening of the structural elements can be carried out. Even if structural safety is assured, it may be necessary to consider repairs to improve the durability of the concrete.

If the analytical assessment of the structure is found to be impractical, it is also possible to carry out a static load test on the structure or structural element in question (described in ACI 318). Since this is often a costly and hazardous procedure, which is also open to interpretation, it is recommended only as a last resort. As with the analytical method, load tests will generally not provide information on the durability of the concrete, which must be assessed separately.

The foregoing procedures are not always followed in practice. Investigations tend to be costly and time consuming, and because of their inherent uncertainties, one is still left in the end to rely on "engineering judgment." However, given the fact that the principles of producing quality concrete are well known, these cases do occur more frequently than one would like to admit.

Although we can never completely eliminate the production of poor concrete, it is nonetheless clear that the present acceptance procedures are inadequate, because they are based largely on 28-day strengths, as discussed in detail in Chapter 14. Much of the trauma of finding poor concrete would be eliminated if more rapid methods of assessing concrete quality were widely adopted. A wider use of accelerated tests for hardened concrete would be helpful.

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PROBLEMS

15.1. Why is concrete an inherently variable material?

15.2. As the project engineer on a large construction site, what remedial action would you take if, on average, one in eight test cylinders fail at less than the specified strength?

15.3. If the specified design strength is 30 MPa, to what average strength should the concrete be designed so that only 1 in 10 test cylinders will fall below the specified strength? The standard deviation is determined to be (a) 1.5 MPa; (b) 3 MPa; (c) 6 MPa.

15.4. As an engineer responsible for the concrete specifications for a structure to be built in a remote area, where there is no experience with local variables, how would you estimate the expected variability of the concrete?

15.5. What strength should be specified in the mix design of a concrete that must attain 40 MPa in 95% of test cylinders given a known standard deviation of 4 MPa?

15.6. How is concrete durability taken into account in concrete quality control?

CHAPTER 16

Time-Dependent Deformation

Concrete will undergo volume changes while in service, due to stimuli such as applied stress, change of moisture content, and changes in temperature. The response of the concrete to these stimuli is complex, resulting in reversible, irreversible, and time-dependent deformations. This chapter deals with three main types of deformations:

1. Shrinkage, which occurs with loss of moisture.
2. Deformations that occur under high rates of loading.
3. Creep, which is the time-dependent deformation that occurs on the prolonged application of stress.

These changes are monitored by measuring longitudinal strain, and these strains are of the same order of magnitude in each case. Strains associated with loading rates typically used in material tests and modulus of elasticity are discussed in Chapter 13, although there is an element of time dependency even in these cases. Strains due to thermal changes are considered in Chapter 17.

16.1 PLASTIC SHRINKAGE

Loss of water from fresh concrete, if not prevented, can cause cracking. The most common situation is surface cracking due to evaporation of water from the surface, but suction of water from the concrete by the subbase or by formwork materials also can cause cracking or can aggravate the effects of surface evaporation. In fresh concrete, the space between particles is completely filled with water. When water is removed from the paste by exterior influences, such as evaporation at the surface, a complex series of menisci are formed. These, in turn, generate negative capillary pressures (see next section), which will cause the volume of the paste to contract. Capillary pressures continue to rise within the paste until a critical "breakthrough" pressure (P_c) is reached, at which point the water is no longer evenly dispersed through the paste and rearranges to form discrete zones of water with voids between. The maximum rate of *plastic shrinkage* occurs just prior to the "breakthrough" pressure, and little shrinkage occurs afterward.

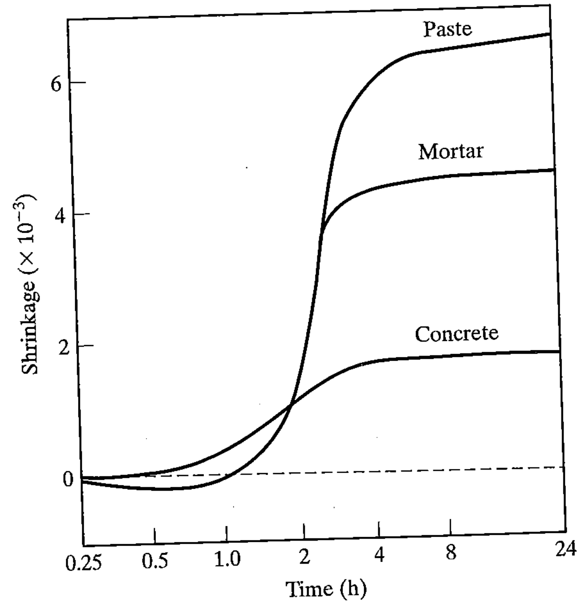


FIGURE 16.1

Effect of aggregate on plastic shrinkage. [From R. G. L'Hermite, *Proceedings, Fourth International Symposium on the Chemistry of Cement*, Washington, DC, Vol. II, pp. 659–694 (1960).]

The effect of aggregate on plastic shrinkage can be seen in Figure 16.1. Plastic shrinkage can in theory be beneficial by causing compaction of the paste; however, in practice, the effects of plastic shrinkage are not uniform throughout the mass, and differential volume changes can cause cracking under induced tensile stresses. Plastic shrinkage cracking (Figure 16.2) is most common on horizontal surfaces of pavements and slabs where rapid evaporation is possible, and its occurrence will destroy the integrity of the surface and reduce its durability. It is aggravated by a combination of high wind velocity, low relative humidity, high air temperature, and high concrete temperatures. These conditions are most prevalent during the summer months, but can occur at any time. If the rate of surface evaporation exceeds 0.5 kg/m^2/h (0.1 lb/ft^2/h), loss of moisture may exceed the rate at which bleed water reaches the surfaces, creating negative capillary pressures, which cause plastic shrinkage. Concretes containing admixtures that reduce the rate of bleeding are particularly susceptible to plastic shrinkage cracking (see Section 11.5).

16.2 DRYING SHRINKAGE

The term *drying shrinkage* is generally reserved for hardened concrete. It represents the strain caused by the loss of water from the hardened material. *Autogenous shrinkage*, which occurs when a concrete can self-desiccate during hydration, is a special case of drying shrinkage. *Carbonation shrinkage*, which occurs when hydrated cement reacts with atmospheric carbon dioxide, also can be considered as a special case of drying shrinkage. Shrinkage is a paste property; in concrete, the aggregate has a restraining influence on the volume changes that will take place within the paste.



FIGURE 16.2

Plastic shrinkage cracking in a concrete pavement. (Photograph courtesy of the Portland Cement Association.)

Parameters Affecting Drying Shrinkage

Drying shrinkage of hardened concrete is a much more important phenomenon than those already described. Inadequate allowance for the effects of drying shrinkage in concrete design and construction can lead to cracking or warping of elements of the structure due to restraints present during shrinkage. The most obvious example is the necessity of providing contraction joints in pavements and slabs. These joints prevent random, irregular shrinkage cracking and confine it to a desired location in a form in which the crack can readily be filled with a sealant to prevent the entry of foreign materials. The response of the paste to moisture loss is modified by the presence of aggregate and the shape of the concrete member. Even with good jointing, uneven shrinkage caused by moisture loss only from the top of the slab can cause curling at the edges. Thus, to understand precisely how different experimental parameters affect the drying shrinkage of concrete, a study of hardened cement paste itself is appropriate. The factors that affect shrinkage are listed in Table 16.1.

Behavior of Cement Paste

The results discussed in this section are based on laboratory work using cement paste specimens with relatively small cross sections. If such specimens are not used, the rate of diffusion of water from the specimen to the surroundings makes equilibrium states difficult to achieve and complicates the interpretation of data. One important aspect

TABLE 16.1 Parameters Affecting Drying Shrinkage and Creep of Concrete

Paste parameters	
Porosity	w/c ratio and degree of hydration
Age of paste	w/c ratio and degree of hydration
Curing temperature	
Cement composition	
Moisture content	
Admixtures	
Concrete parameters	
Aggregate stiffness	
Aggregate content (cement content)	
Volume–surface ratio	
Thickness	
Environmental parameters	
Applied stress	affect only creep
Duration of load	affect only creep
Relative humidity	
Rate of drying	
Time of drying	

concerning the drying shrinkage of cement paste, and hence of concrete, is the fact that part of the total shrinkage that occurs on the first drying is irreversible (see Figure 16.3). Thus, the subsequent volume expansions that occur on rewetting and the volume contractions that occur on subsequent drying are smaller. Experimental parameters generally have some effect on the amount of irreversible shrinkage as well as on total shrinkage. This improvement in dimensional stability after first drying can be used to advantage in precast concrete products.

Since moisture loss is the underlying cause of drying shrinkage, the relationship between the two should be of interest. Figure 16.4 shows a typical shrinkage–weight loss curve for cement paste in which five domains are observed. Domain 1 is attributed to loss of water from large capillary pores (macropores). Within Domain 2, water is lost concomitantly from both mesopores and micropores (i.e., water is lost from both the finer capillary pores and the intrinsic C–S–H porosity). At the end of Domain 2, all water held in the pore system has been removed. During Domains 3 and 4, water adsorbed on solid surfaces is removed, while in Domain 4, the interlayer water of C–S–H is also removed. All of this water can be removed at room temperature (evaporable water) or more quickly at 105°C (221°F). Above this temperature, decomposition of C–S–H is responsible for the additional shrinkage observed in Domain 5. The closely related plot of shrinkage versus relative humidity (Figure 16.5) emphasizes the differences between the various domains. Since in most civil engineering applications concrete is not exposed to very low relative humidities for extended periods, we need only concern ourselves with behavior in the upper humidity range. Shrinkage in Domains 4 and 5 will only occur in concrete exposed to fire.

Mechanisms of Reversible Shrinkage Three phenomena are believed to contribute to bulk shrinkage of cement paste: capillary stress, disjoining pressure, and changes in

surface free energy. These phenomena result from the special nature of hydrated cement paste: its high porosity with a network of small capillary pores, the extensive van der Waals' bonding in C–S–H, and the high surface area and intrinsic microporosity

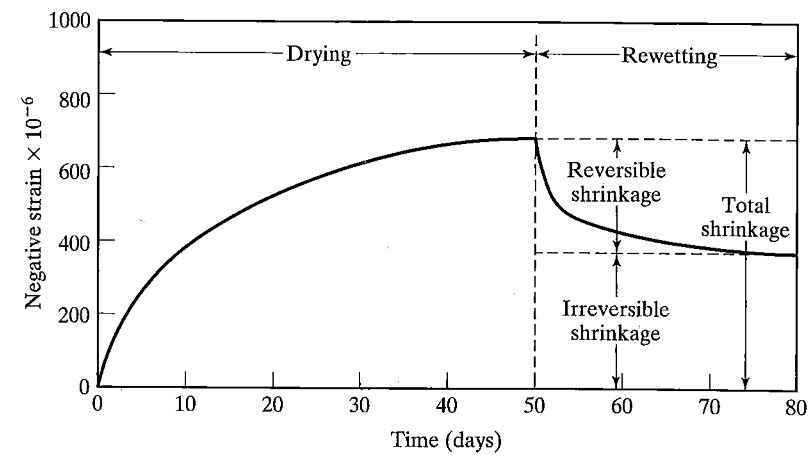


FIGURE 16.3

Typical behavior of concrete on drying and rewetting.

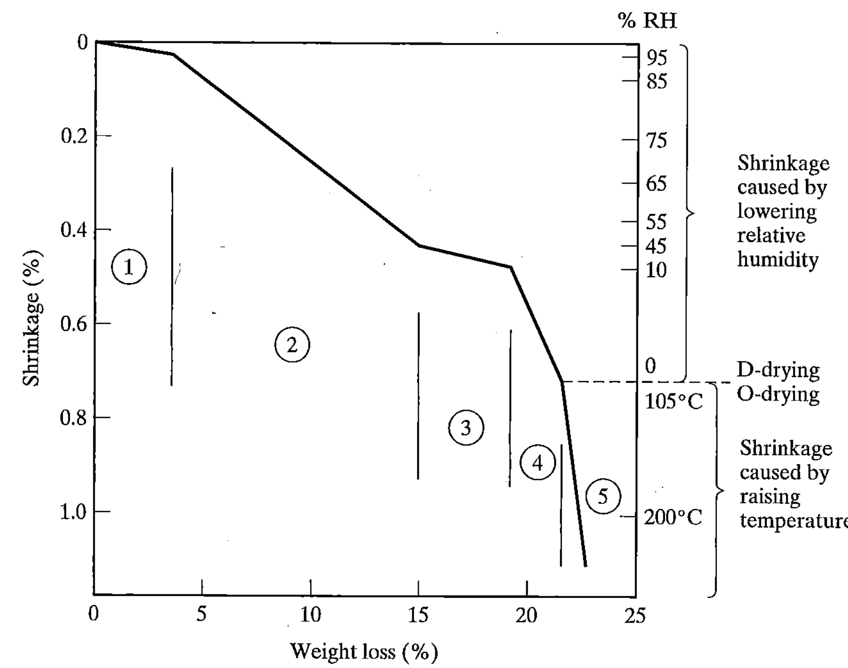


FIGURE 16.4

Schematic representation of shrinkage–water loss relationships for cement pastes during drying.

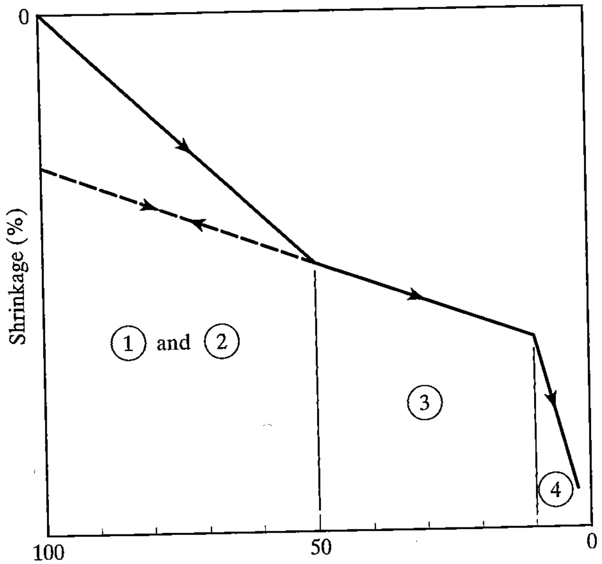


FIGURE 16.5

Schematic representation of drying shrinkage of cement paste during drying.

of C–S–H. Thus, the net linear shrinkage is a function of internal pressures developed by capillary stresses P_{cap}, disjoining effects P_{dis}, and changes in surface free energy P_{sfe}. These effects are illustrated schematically in Figure 16.6. Although it is linear shrinkage that is usually measured experimentally, a volumetric contraction actually occurs. P_{cap}, P_{dis}, and P_{sfe} are dependent on both relative humidity and temperature.

Capillary Stress. Water within a small capillary pore is partially under the influence of surface interactions exerted by the pore walls. Therefore, the water cannot be removed by evaporation unless the relative humidity (RH) is lowered by an amount depending on the pore radius (r) and given by the equation

$$\ln(\mathrm{RH}) = K\left(\frac{2\gamma}{r}\right) \tag{16.1}$$

where γ is the surface free energy (surface tension) of the water, and K is a constant. The water can only be lost by the creation of an air–water interface (meniscus) which requires energy. As the result of the interactions with the pore walls, the meniscus adopts a curved surface, and the water is in a state of hydrostatic tension (P_{cap}), where

$$P_{cap} = \frac{2\gamma}{r} \tag{16.2a}$$

$$= \frac{\ln(RH)}{K} \tag{16.2b}$$

The water exerts a corresponding hydrostatic compression on the solid skeleton, and if internal particle rearrangement can occur (the case for cement paste), some pores can become smaller.

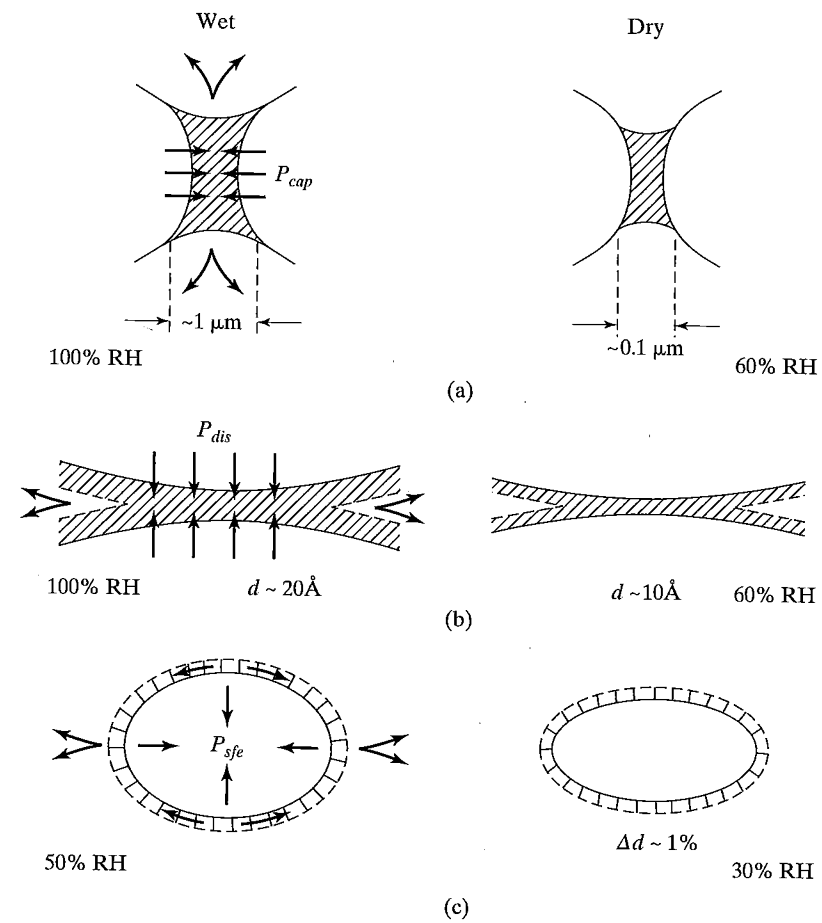


FIGURE 16.6

Proposed mechanisms for causes of drying shrinkage of cement paste: (a) capillary stress; (b) disjoining pressure; (c) surface tension.

The difference between Domains 1 and 2 in Figures 16.4 and 16.5 is thus simply explained by the fact that the larger capillaries, which are emptied at relative humidities down to about 95%, have a relatively large volume-to-surface ratio and develop very small stresses. At lower humidities, although the capillary volume becomes much smaller, stresses rise quite rapidly. Capillary stresses cannot exist below about 45% RH since menisci are no longer stable. The removal of hydrostatic stress at this relative humidity should be accompanied by a relaxation in the solid and an *increase* in length. Such behavior is observed in materials with a rigid skeleton (e.g., porous glass), but is obscured in cement paste by the slower concomitant loss of water from micropores (subsequently discussed).

Disjoining pressure. Water is adsorbed on the surfaces of C–S–H at all relative humidities, the thickness increasing with increasing humidity. An assembly of colloidal

particles, as in C–S–H, has van der Waals' forces attracting adjacent particles and bringing their adjacent surfaces in close contact. Adsorption of water between the C–S–H surfaces creates a disjoining pressure (which results from the orientation of water molecules in the adsorbed film). The disjoining pressure increases with the increasing thickness of the adsorbed water between particles (i.e., increasing relative humidity) until it exceeds the van der Waals' attractions, when the particles will be forced apart, creating a dilation. C–S–H is formed in the dilated state during hydration and the water-filled spaces are the micropores. Therefore, on first drying, the decreasing disjoining pressure, which accompanies a lowered relative humidity, causes the particles to be drawn together by the van der Waals' forces, and there is a net volume decrease. Disjoining pressure is only a significant factor down to about 45% RH, and it shows an RH dependency similar to capillary stress.

Surface free energy. Below 45% RH, when capillary stress and disjoining pressure are no longer operating, shrinkage is explained by changes in surface energy. As the most strongly adsorbed water (equivalent to one or two molecular layers) is removed, the surface free energy of the solid begins to increase significantly. It is well known that a liquid drop is under hydrostatic pressure by virtue of its surface tension (surface free energy). This pressure is described by Eq. (16.3), because the drop is bounded by a meniscus. A solid particle is likewise subjected to a mean pressure

$$P_{sfe} = \frac{2\gamma S}{3} \tag{16.3}$$

where γ is the surface energy in J/m^2 and S the specific surface area of the solid in m^2/g. Since S is large (~400 m^2/g) in the case of C–S–H, P_{sfe} can be large and causes compression in the solid. The change in solid volume with changing γ has been observed for other materials and was first determined in experiments on powdered coal conducted by Bangham. It can be described by the Bangham equation

$$\frac{\Delta l}{l} = k\Delta\gamma \tag{16.4}$$

where $\Delta l/l$ is the observed relative length change, $\Delta\gamma$ is the change in surface free energy during adsorption, and k is a constant of proportionality. $\Delta\gamma$ is dependent on ln(RH); but in the range 20 to 45% RH, the change is approximately linear. Above 45% RH, changes in γ can be neglected, while the greatest effect is observed below 20% RH.

Irreversible Shrinkage It can be seen from Figure 16.3 that a considerable part of the observed length change on first drying is irreversible, although the mechanisms described would predict reversible behavior. Figure 16.5 shows that irreversibility occurs only on first drying and that deformations on subsequent wetting and drying cycles are essentially reversible. It has been demonstrated that increases in drying shrinkage that attend the use of some admixtures are due to increases in the irreversible component on first drying and not the reversible portion. Furthermore, Figure 16.7 shows that total shrinkage on first drying increases as the porosity of the paste increases. If swelling on

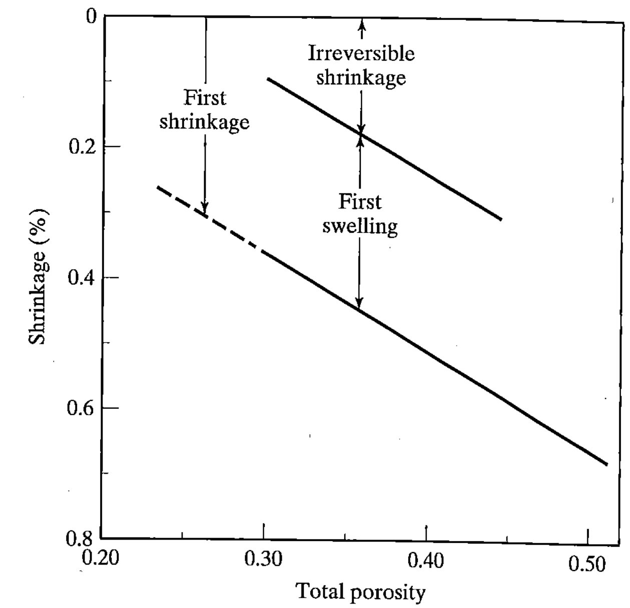


FIGURE 16.7
Effect of porosity on the irreversible drying shrinkage of cement paste. [After R. A. Helmuth and D. H. Turk, *Journal of the Portland Cement Association, R & D Laboratories*, Vol. 9, pp. 8–21 (1967).]

rewetting is measured, it is found that only the irreversible component is affected by porosity. Since the *w/c* ratio and degree of hydration control porosity, these parameters also affect irreversible shrinkage. These facts can be explained in part by changes in the distribution of pores within the paste. On first drying, the permeability of the paste increases nearly twofold, and this has been taken as evidence that a continuous network of capillaries is formed. The development of capillary stress is then much reduced.

The temperature of curing also influences the drying shrinkage of cement pastes. If pastes are exposed to elevated temperatures during moist curing, irreversible shrinkage is reduced, but reversible shrinkage is unaffected. The decrease in shrinkage depends on the maximum temperature to which the paste is exposed; at 65°C (150°F), irreversible shrinkage can be reduced by about two-thirds and total shrinkage by about one-third. This reduction is due to a larger proportion of the capillary porosity forming as macropores and a reduced microporosity of C–S–H. The effect depends on the length of time the paste is maintained at higher temperatures. However, the time of exposure at high temperatures that is required to reduce shrinkage can be relatively short and may be less than the total specified curing time. Even exposure to higher temperatures after curing is finished can reduce irreversible shrinkage.

Irreversibility also is strongly affected by the drying history of the paste. If the paste is dried quite slowly, by being conditioned at progressively lower relative humidities, total shrinkage is less than if the paste is dried directly to the lowest relative humidity. This is an important observation, since drying of concrete can be a relatively slow process in thick sections where diffusion of water from the concrete controls the rate of moisture loss. The time at which the paste is held at a low relative humidity also has a strong influence on irreversible shrinkage (see Figure 16.8). The slow, continued

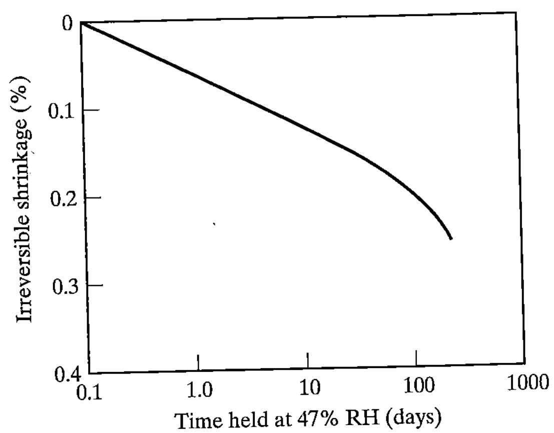


FIGURE 16.8

Effect of drying time on irreversible shrinkage of cement paste. [Based on data by R. A. Helmuth and D. H. Turk, *Journal of the Portland Cement Association, R & D Laboratories*, Vol. 9, pp. 8–21 (1967).]

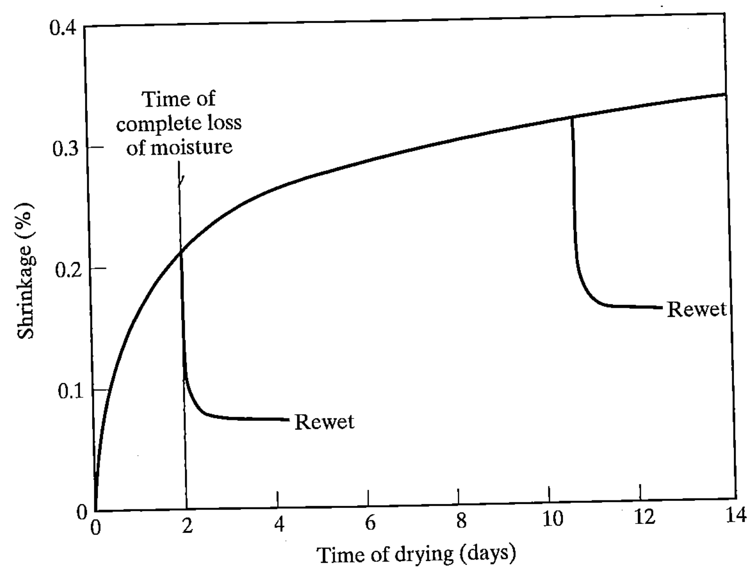


FIGURE 16.9

Increase of total and irreversible drying shrinkage of cement paste with drying time.

increase in total shrinkage that has been observed after all of the water has effectively been removed from the specimen is almost all due to an increase in irreversible shrinkage (see Figure 16.9).

The origins of the irreversible strains can be found in the unstable amorphous nature of C–S–H. As water is removed from the micropores and capillary stress is applied to the random assemblage of C–S–H particles, rearrangement of their packing will occur. These changes can lead to an alteration in the bonding between sheets (e.g., creation of new interlayer spaces) and creation of new mesopores (or enlargement of mesopores) due to annihilation of micropores. These changes are analogous to those promoted by prolonged hydration or high curing temperatures and can be considered as "aging" of the C–S–H accelerated by drying. Changes in *w/c* ratio and the addition of chemical or mineral admixtures can also alter the density of packing of C–S–H particles during their formation. This in turn affects the bonding between them, the size distribution of the mesopores, and the subsequent "aging" on drying. Exposure to high-temperature curing apparently increases the amount of chemical bonding between C–S–H particles. The aging process, which is independent of hydration, is both time and temperature dependent and stabilizes the C–S–H so that it is less deformed on loss of moisture. In addition to these fundamental changes to the microstructure, some of the irreversible strain may be the result of microcracking. Humidity gradients that develop during drying will give rise to differential shrinkage and impose tensile stress at the surface of the specimen. This situation will be particularly applicable to concrete and will be discussed again later.

Cement Composition The composition of the cement can influence the shrinkage of the paste, and hence of the concrete, although the effect is not large and the exact relationships between composition and shrinkage are not well understood. Correlations have been found between shrinkage and C_3A content, from which it might be concluded that the sulfoaluminates contribute to shrinkage. There is also an "optimum gypsum content" for minimum shrinkage for each cement. It has recently been observed that the shrinkage of pastes of pure C_3S and C_2S is less than that of a portland cement paste. Since the bulk of the shrinkage is believed to originate in the C–S–H and its associated porosity, it would seem that in cement pastes, the presence of minor components changes the properties of C–S–H and the mesopore structure. Calcium hydroxide apparently has little effect on shrinkage.

Factors Influencing Shrinkage

Effect of Aggregates The drying shrinkage of concrete will be less than that of pure paste because of the restraining influence of aggregate. With rare exceptions, aggregates are dimensionally stable under changing moisture conditions. The amount of restraint provided by aggregate depends on the amount of aggregate in the concrete (Figure 16.10), its stiffness (Figure 16.11), and the maximum size of the coarse aggregate. The stresses at the cement paste–aggregate interface due to drying shrinkage increase as the maximum aggregate size increases. These higher internal stresses will increase the amount of cracking in the interfacial region. Lightweight aggregates are generally dimensionally stable, but their low modulus of elasticity means that lightweight concretes can be expected to have higher shrinkages than normal-weight concretes. The combined effects of aggregate content and stiffness are embodied in the empirical equation

$$\varepsilon_{con} = \varepsilon_p(1 - V_a)^n \tag{16.5}$$

where ε_{con} and ε_p are the shrinkage strains of concrete and paste, respectively; V_a is the volume fraction of aggregate; and n, which varies between 1.2 and 1.7, depends on the elastic properties of the aggregates.

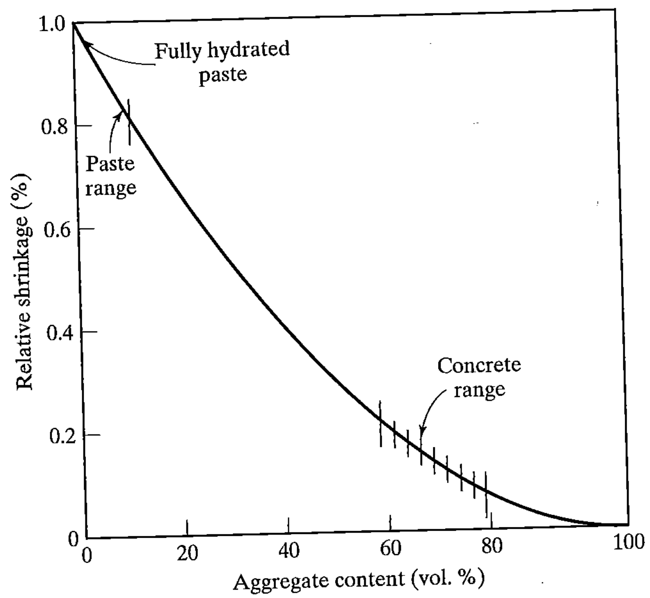


FIGURE 16.10
Influence of aggregate content on the drying shrinkage of concrete.

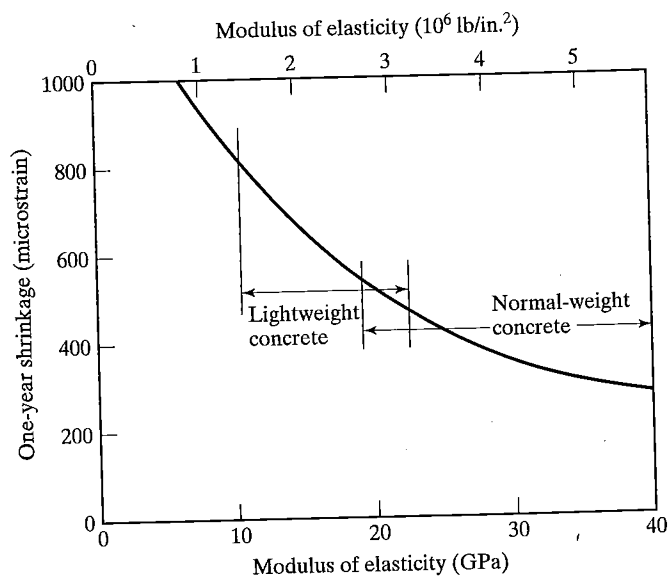


FIGURE 16.11
Effect of aggregate modulus on the drying shrinkage of concrete.

Specimen geometry The size and shape of a concrete specimen will determine the rate of moisture loss and hence the rate and magnitude of drying shrinkage. The length of the diffusion path has a strong influence on the rate of moisture loss. In mass concrete, it will take approximately 1 month for the outer 75 mm (3 in.) of concrete to reach moisture equilibrium, 1 year for 225 mm (9 in.), and 10 years for 600 mm (24 in.) to reach equilibrium. Thus, volume-to-surface area ratios will be important, and a higher ratio will lead to less shrinkage. A T-beam section will dry more rapidly than a square beam, but have less ultimate shrinkage. Rate of moisture loss depends on total surface area and the average length of the diffusion path. The smaller the initial cross section of a member, the faster the initial rate of shrinkage, but the lower the magnitude of shrinkage at later times. Similarly, with reduced surface areas, the lower rates of early shrinkage extrapolate to large ultimate shrinkage. Thus, there is an inverse relationship between rate of early shrinkage and ultimate shrinkage.

This is opposite to the effect observed for pastes. This apparent contradiction is, in part, due to the increased amount of hydration of the slower-drying specimens, but is primarily caused by restrained shrinkage. During drying of a concrete specimen, the situation soon arises where a dry, contracting exterior surrounds a still-moist core that has not begun to shrink (see Figure 16.12). Thus, tensile stresses will be set up in the outer part of the prism, and microcracking is likely to occur unless the stress is relieved by tensile creep. Either creep or microcracking will act to reduce the observed shrinkage. Furthermore, the moist core will be subject to a compressive stress under which it will creep, and this may also affect the magnitude of later drying shrinkage.

Autogenous Shrinkage

If no additional water beyond that added during mixing is provided during curing, concrete will begin to dry internally, even if no moisture is lost to the surroundings as water is consumed by hydration. However, bulk shrinkage is only observed in concretes with a low *w/c* ratio (<0.3) and is increased by the addition of reactive pozzolans (e.g., silica fume).

The phenomenon is known as *self-desiccation* and is manifested as *autogenous shrinkage* (also known as *chemical shrinkage*). In extreme cases, the internal relative

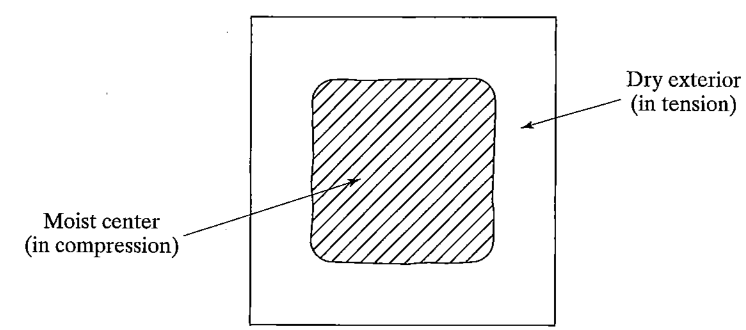


FIGURE 16.12
Restraint of shrinkage in a partially dried concrete member.

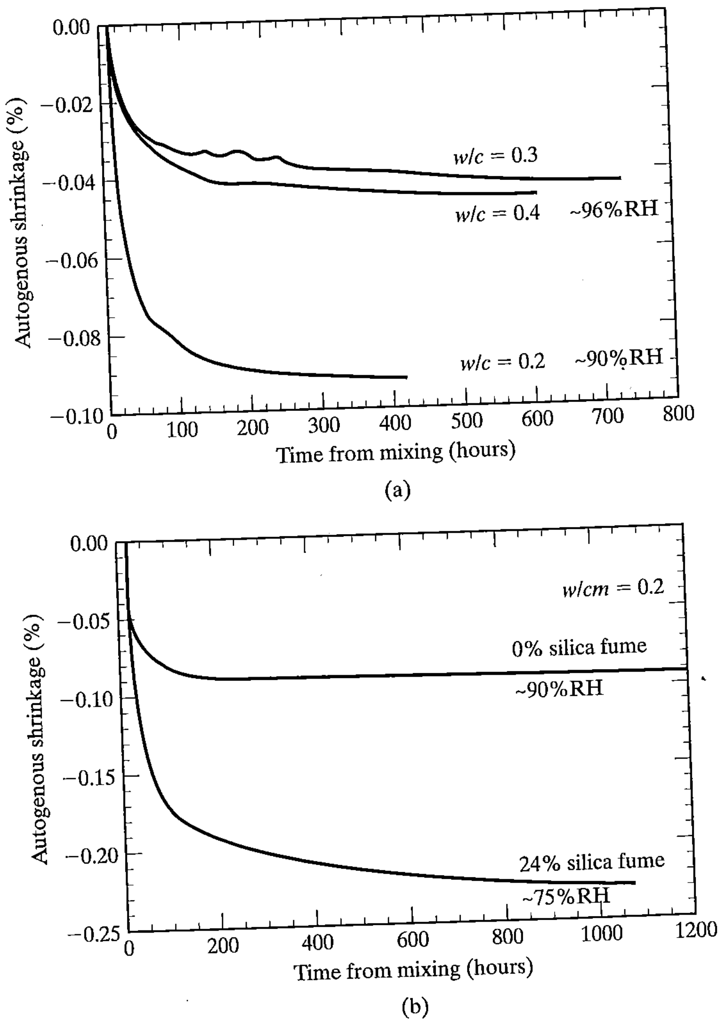


FIGURE 16.13

Autogenous shrinkage of cement paste: (a) effect of *w/c* ratio; (b) effect of adding silica fume. Approximate internal RH is given in each case. [Based on data by H. Ai, Ph.D. Thesis, University of Illinois at Urbana-Champaign (2000).]

humidity can drop to 75–80% RH. Thus, this shrinkage is a special case of drying shrinkage (and of similar magnitude), since it is immaterial whether the water is removed by physical or chemical processes. Autogenous shrinkage will only occur if the concrete is sealed or in dense concretes, e.g., low *w/c* and additions of silica fume (see Figure 16.13). In the latter case, self-desiccation may occur to some extent, even if water is supplied during the curing process, because external water cannot easily penetrate the concrete. Otherwise, any effects of self-desiccation are usually masked by expansion associated with the formation of ettringite or the hydration of free MgO.

Carbonation Shrinkage

Hardened cement paste will react chemically with carbon dioxide. The amount present in the atmosphere (~0.04%) is sufficient to cause considerable reaction with cement paste only over a long period of time. However, this is accompanied by irreversible

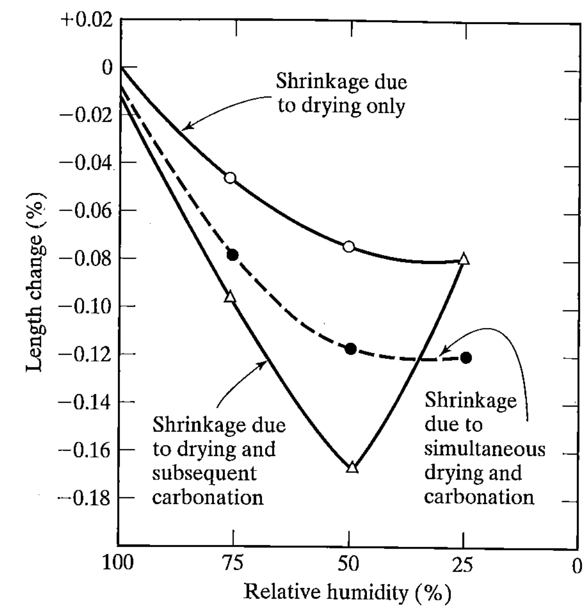


FIGURE 16.14

Shrinkage of mortar bars during carbonation. [After G. J. Verbeck, in *Papers on Cement and Concrete*, ASTM STP 205, pp. 17–36 (1958).]

shrinkage, and hence it is called *carbonation shrinkage*. The extent to which cement paste can react with carbon dioxide, and hence undergo carbonation shrinkage, is a function of relative humidity (see Figure 16.14) and is greatest around 50% RH. At high humidities, carbonation is low because the pores are mostly filled with water and CO_2 cannot penetrate into the paste very well. At very low humidities, an absence of water films is believed to lower the rate of carbonation. Carbonation shrinkage is greatest when carbonation occurs after drying, rather than during drying, except at low humidities.

Concrete exposed to carbonation loses water and behaves as though it has been dried to a much lower relative humidity than that to which it is actually exposed. The shrinkage–water loss relationship is similar to that observed for normal drying. Furthermore, carbonation shrinkage is wholly irreversible. It is believed that CO_2 reacts with C–S–H inducing a decrease in its C/S ratio and a concomitant loss of water.

$$\text{C–S–H} + \overline{\text{C}} \rightarrow \text{C–S–H} + \text{C}\overline{\text{C}} + \text{H} \qquad (16.6)$$

Carbonation of C–S–H is known to change the bonding characteristics of the material, which could account for the irreversible nature of the accompanying shrinkage. Thus, carbonation can be viewed as promoting changes in C–S–H that normally only occur at much lower relative humidities. Calcium hydroxide also will form calcium carbonate by reacting with atmospheric CO_2.

Carbonation shrinkage can be important from a practical point of view. Advantage can be taken of its irreversible nature for precast concrete. For example, by exposing concrete block to CO_2-rich air (to hasten the carbonation process), the block can be made much more dimensionally stable to subsequent wetting and drying. On the other hand, carbonation can be detrimental to cast-in-place concrete. Since it is much

less porous than block, carbonation can occur only near the outside, precisely where the maximum rate of drying is also occurring. Thus, carbonation shrinkage is likely to be maximized and, when added to drying shrinkage, may cause severe shrinkage cracking. Concrete may be damaged by carbonation in the winter months when it is cured in enclosed areas that are heated with oil burners, due to higher levels of CO_2 in improperly vented enclosures. In addition, plastic shrinkage may occur due to rapid drying out if a high relative humidity is not maintained. Such a condition can lead to crazing or dusting of the surface. In the long term, the slightly higher levels of CO_2 that can exist in enclosed parking garages may eventually lead to carbonation damage and increased penetration of chloride ions. Carbonation is more extensive the greater the surface area, and in thin specimens, carbonation shrinkage may become a significant fraction of the total shrinkage.

16.3 STRAIN RATE EFFECTS

The behavior of concrete in both tension and compression is dependent on the rate of load application. Thus, there is no aspect of the stress–strain response of concrete that is truly independent of strain rate. The strain rate sensitivity of concrete (Figure 16.15) is usually ascribed to two causes: the behavior of cement paste as a viscoelastic material controlled by the viscous time-dependent movement of free water through voids and pores, and the time-dependent nature of crack growth relative to the load rate. The importance of moisture is, unfortunately, not always recognized in studies aimed at measuring the strain rate sensitivity of concrete, resulting in a wide scatter in the reported results. However, like the creep response of concrete (discussed in Section 16.4), the strain rate sensitivity of concrete and its mortar and cement paste constituents is greatest when the materials are fully saturated and least when the materials are dry. This point is demonstrated by the two curves in Figure 16.15a. As shown in the figure, dry concrete exhibits very little strain rate sensitivity, except at very high strain rates (far above what is encountered by most structures).

Crack growth plays a role in strain rate response, because cracks require a finite time to propagate. Crack velocity increases with increasing strain rate, but remains only a small fraction of the stress wave velocity in concrete. At high rates of loading, when cracks propagate much more slowly than the applied stress, the crack path is altered and the crack length is shortened, which translates into improved compressive, tensile, and fracture properties at increasing load rates.

When discussing the rate sensitivity of concrete and its constituents, it should be noted that, due to the nonlinear stress–strain behavior of the materials, strain rate responses differ depending upon whether the application of load is controlled by stress or displacement. These differences, however, are usually small, since significant changes in material response (for example, a 10 to 15% change in compressive strength) occur only for differences in loading rate on the order of 10 times or more.

Compression

In compression, wet concrete exhibits a 7 to 27% increase in compressive strength with each factor of 10 increase in strain rate, up to a strain rate of about 10 sec^{-1} (Figure 16.15b). (Note that, since strain is a unitless quantity, the rate of change in

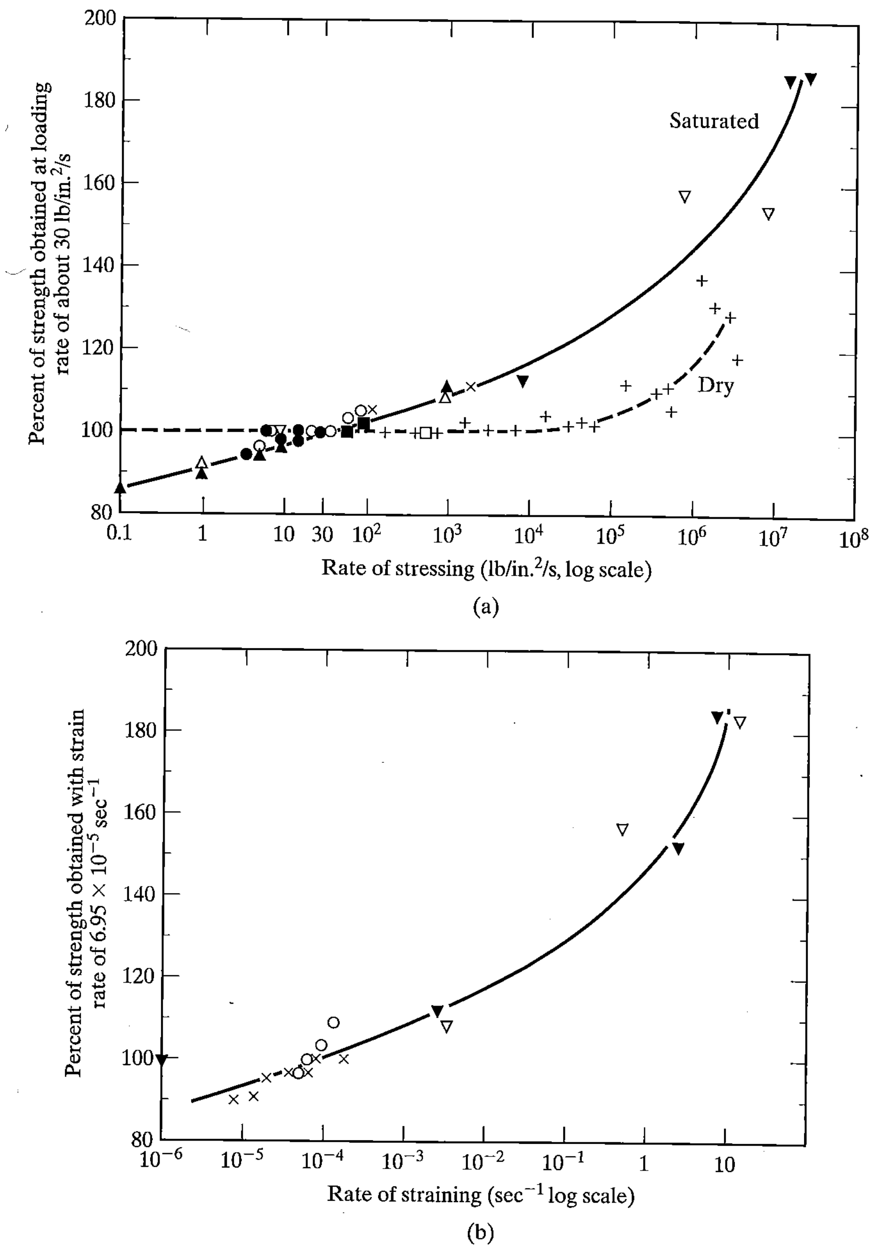


FIGURE 16.15

Influence of (a) stress rate and (b) strain rate on the compressive strength of concrete. [From D. McHenry and J. J. Shideler, in *Symposium on Speed of Testing of Nonmetallic Materials*, ASTM STP 185, pp. 72–82 (1955). Copyright ASTM INTERNATIONAL. Reprinted with permission.

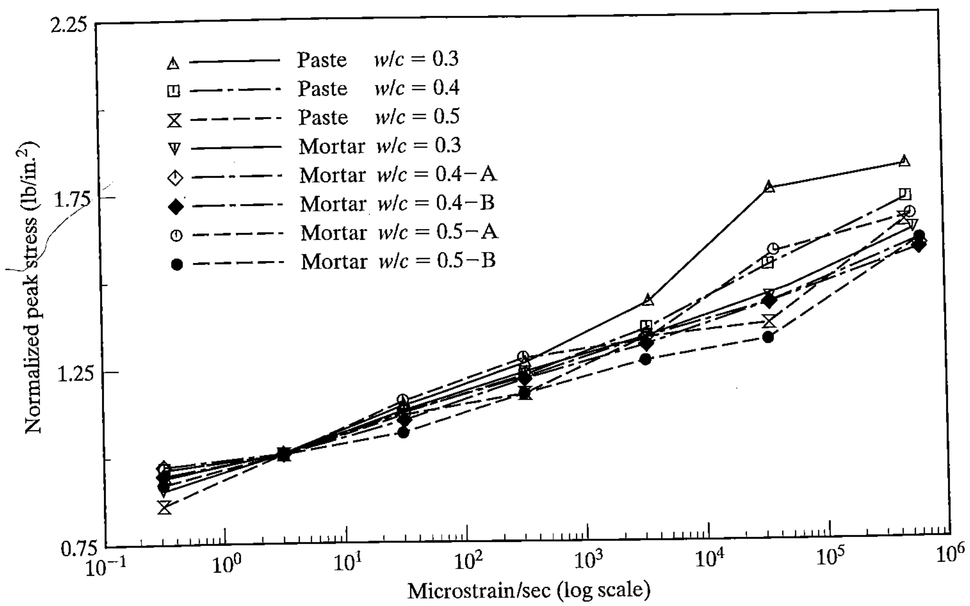


FIGURE 16.16

Normalized peak stress versus strain rates for cement paste and mortar. [From S. Harsh, Z. Shen, and D. Darwin, *ACI Materials Journal*, Vol. 87, No. 6, pp. 508–516 (1990).]

strain is expressed in units of sec^{-1}). Similar strain rate sensitivity is exhibited by the cement paste and mortar constituents of concrete, as shown in Figure 16.16. (Note the log scale for strain rate.) Dry concretes invariably exhibit less than a 5% increase in strength with each factor of 10 increase in strain rate. At strain rates above 10 sec^{-1}, the strain rate sensitivity appears to increase (Figure 16.15b) and then to level off, as shown in Figure 16.17 for cement paste.

The strain corresponding to the peak stress obtained in a compressive test is highest at the lowest rates of loading, principally due to the effects of creep. The strain corresponding to the peak stress first decreases (as the effects of creep are reduced) and then increases as the strain rate is increased. This behavior, which is demonstrated for mortar in Figure 16.18, can be explained by the viscoelastic nature of cement paste and by the effects of strain rate on microcracking in concrete. At low strain rates, below about 10^{-6} sec^{-1}, the cement paste constituent of concrete deforms over time as creep takes place. The effects of moisture movement and other aspects of the creep response play a rapidly decreasing role at higher strain rates, with the cement paste constituent of concrete behaving more and more like a linear, elastic, brittle material. At higher rates of loading, an increasingly greater portion of the strain is accounted for by microcracking. While bond cracking between the coarse aggregate and surrounding mortar appears to be largely unchanged for a given applied strain, mortar cracking and microcracking within the cement paste constituent of concrete increase measurably with increasing strain rate.

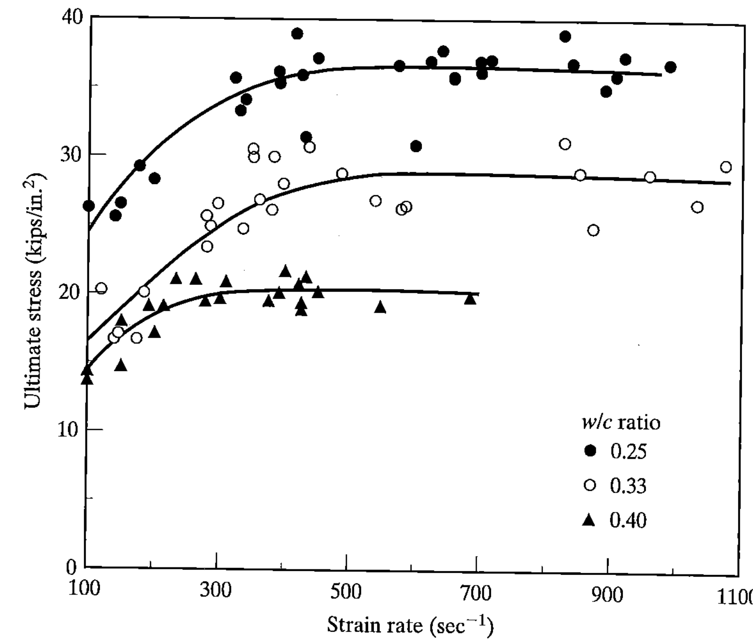


FIGURE 16.17

Ultimate stress versus strain rate for 28-day-old cement pastes at high strain rates. [From I. Jawed, G. Childs, A. Ritter, and S. Winzer, *Cement and Concrete Research*, Vol. 17, No. 3, pp. 403–440 (1987).]

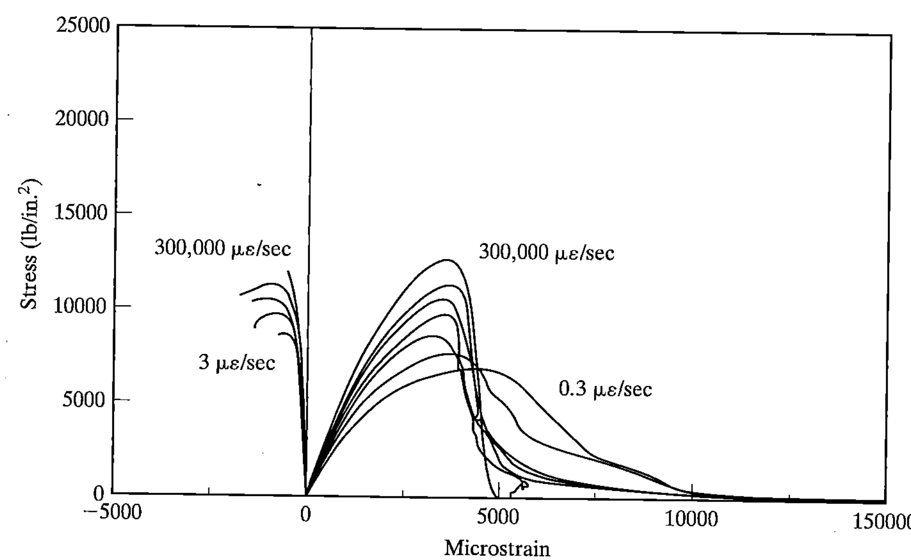


FIGURE 16.18

Stress versus longitudinal and transverse strain for cement paste with w/c ratio = 0.4 at strain rates from 0.3 to 300,000 $\mu\varepsilon$/sec. [From S. Harsh, Z. Shen, and D. Darwin, *SM Report*, No. 22, University of Kansas Center for Research, Lawrence, KS (1989).]

As shown in Figures 16.18 and 16.19, the modulus of elasticity also increases with the increasing strain rate. The increase is about 7% for each factor of 10 increase in strain rate or about half the rate observed for compressive strength. Poisson's ratio also increases with strain rate up to about 80% of the compressive strength and then decreases at higher stresses.

The sensitivity of concrete to strain rate depends upon the aggregate properties. Concretes containing angular, rough, or low stiffness aggregates exhibit more strain rate sensitivity than concretes containing round, smooth, or high stiffness aggregates, respectively. Younger concretes exhibit less rate sensitivity at low strain rates, but more rate sensitivity at higher strain rates, than do older concretes. The relatively low strain rate sensitivity of younger concretes at low stress rates may be due to the open nature of cement paste at early ages, which would tend to make the flow of water easier. The higher strain rate sensitivity of younger concretes at high stress rates may be due to their high free water content, which plays a significant role once the stress rate is high enough.

Static Fatigue

Low initial strain rates up to 25 to 30% of the compressive strength, followed by a high strain rate, result in a higher compressive strength. This observation matches those made for other loading regimes, as discussed in Chapter 13, which indicate that the application of low levels of stress over an extended period of time will increase the strength of concrete. If, on the other hand, load is applied slowly, over a long period of time, concrete will fail at a lower strength than attained in a standard test. This phenomenon is known as *static fatigue*. As shown in Figure 16.20, concrete that is subjected to a

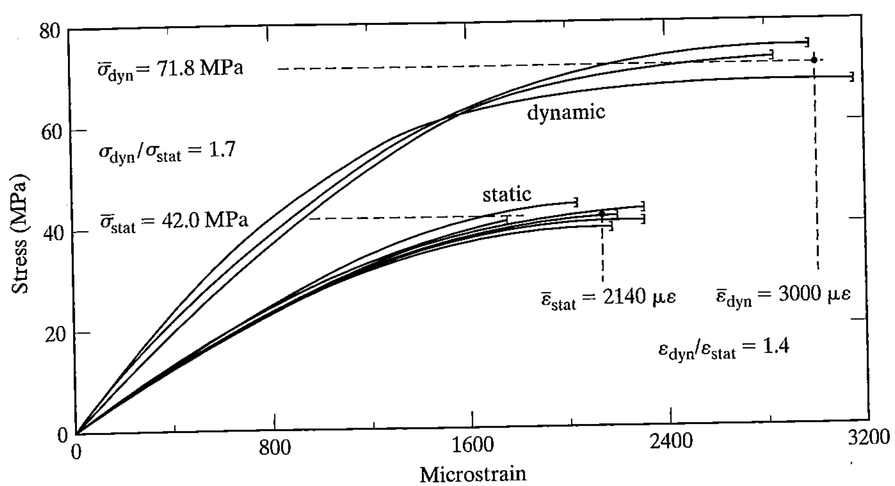


FIGURE 16.19

Stress–strain curves for static and dynamic compressive loading of concrete. [Adapted from M. Cyrback and J. Eibl in *International Conference on Recent Developments in Fracture of Concrete and Rock*, Elsevier Applied Science Press, London and New York, pp. 193–202 (1989).]

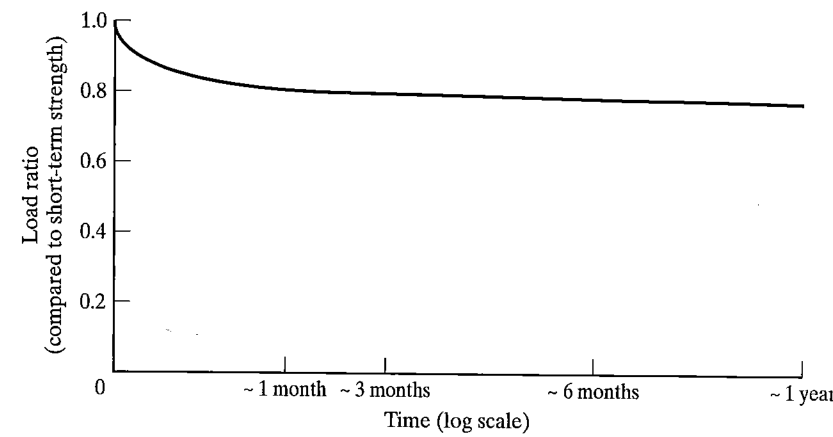


FIGURE 16.20

Static fatigue of concrete (schematic).

sustained load of about 75% or more of the short-term static strength will eventually fail. Under sustained loads, the cement paste constituent of the concrete continues to deform and microcracks continue to grow. Eventually, these cracks reach some critical size, and fracture occurs. Under this type of loading, the strain at failure is higher than in short-term tests because of both the greater deformation of the cement paste and the increased quantity of cracks. At lower sustained stresses, the cracking that does occur is compensated for by the consolidation of the cement paste and an accompanying increase in strength. The crack extension that occurs under sustained loading is sensitive to the presence of water. As a result, dry concrete specimens take much longer to fail under sustained load than do saturated specimens.

Tension

The tensile behavior of concrete may involve the response of members that are either initially cracked or initially uncracked. In either case, the strain rate sensitivity of the peak load appears to be similar to that observed for compressive strength. Tensile strength of a cross section can be predicted by using the Mihashi and Wittmann[1] power model

$$\frac{f_t}{f_{t0}} = \left(\frac{\dot{\sigma}}{\dot{\sigma}_0}\right)^{\frac{1}{1+\beta}} \tag{16.7}$$

in which f_{t0} is the quasi-static tensile strength measured at a low stress rate $\dot{\sigma}_o$, f_t is the tensile strength at stress rate $\dot{\sigma}$, and β is a material parameter (ranging from 19 to 27 for structural concretes). For $\beta = 23$, f_t increases by about 10% for each 10-fold increase in the stress rate $\dot{\sigma}$. This value is closely matched by the observation that the modulus of rupture obtained from Charpy specimens increases by 100% as the strain rate increases from 10^{-6} sec^{-1} to 1 sec^{-1}. Figure 16.21 shows a load versus crack mouth

[1]H. Mihashi and F. H. Wittmann, "Stochastic Approach to Study the Influence of Rate of Loading on Strength of Concrete," *Heron*, Vol. 25, No. 3 (1980).

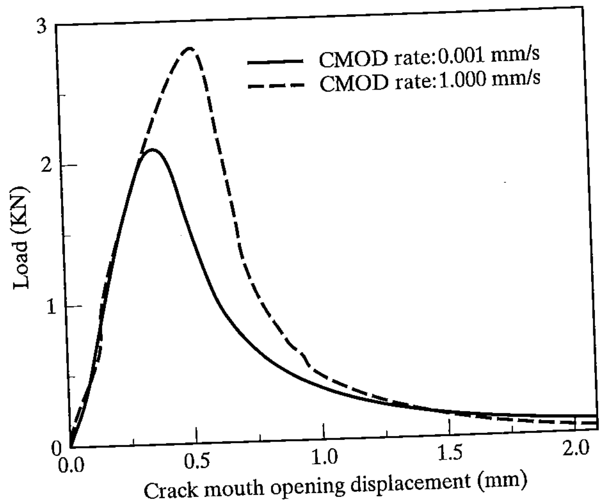


FIGURE 16.21

Load vs. crack mouth opening displacement curves for different loading rates. [From V. Slowik, G. A. Plizzari, and V. E. Saouma, *ACI Materials Journal*, Vol. 93, No. 3, pp. 272–283 (1996).]

opening displacement (CMOD) curve for two fracture specimens for which the CMOD rate differs by a factor of 1000. This increase in load rate results in a 34% increase in the peak load, corresponding to a 10% increase in strength for each factor of 10 increase in the load rate.

Tensile tests at stress rates ranging from 0.1 to 3.0×10^4 MPa/sec (1.5 to 4.4×10^6 lb/in.^{2}/sec) indicate increases in tensile strength of 33 to 134%. In these tests, at the lowest stress rate, cracks pass around aggregate particles, while at stress rates of 3.0×10^3 and 3.0×10^4 MPa/sec (4.4×10^5 and 4.4×10^6 lb/in.^{2}/sec), the cracks are considerably straighter, passing through many aggregate particles. Increasing the stress rate results in higher stresses at fracture due to crack propagation through shorter regions of higher resistance. This is accompanied by an increase in the strain corresponding to the peak tensile stress, due principally to an increase in the amount of microcracking. Figure 16.22 illustrates the relationship between fracture strain and strain rate. In terms of crack behavior, a change in the rate of loading affects the size of the fracture process zone at the tip of a crack. This zone appears to be largest in specimens loaded under quasi-static conditions (strain rate of 10^{-6} sec^{-1}), decreasing both for high rates of loading and loading under creep conditions.

As observed for concrete in compression, weaker concretes tend to be more strain rate sensitive in tension than higher-strength concretes. This is demonstrated for tensile strength in Figure 16.23 and for the critical stress intensity factor in Figure 16.24. In fact, while K_{Ic} (static loading) is higher for high-strength concrete than for normal-strength concrete, K_{Id} (dynamic loading) is actually lower for high-strength concrete that for normal-strength concrete at high rates of loading for the drop weight impact test results shown in Figure 16.24. Also, as observed for concrete in compression, the modulus of elasticity of wet concrete in tension increases with increasing strain rate. In one study, the modulus increased by 50% as the strain rate increased from 4.9×10^{-4}/sec to 0.24/sec.

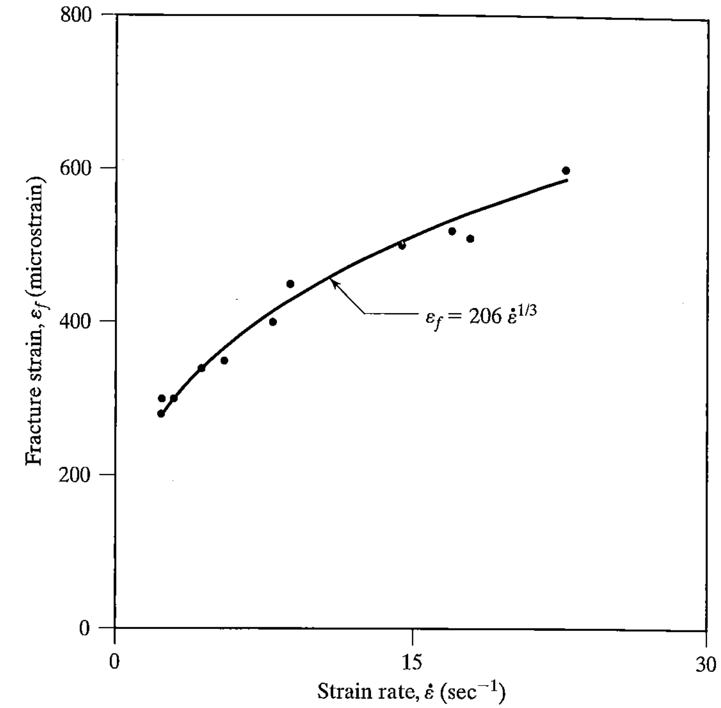


FIGURE 16.22

Fracture strain in tension versus strain rate. [Based on data from D. L. Birkimer and R. Lindemann cited in *Journal of the American Concrete Institute*, Vol. 68, No. 1, pp. 49–57 (1971).]

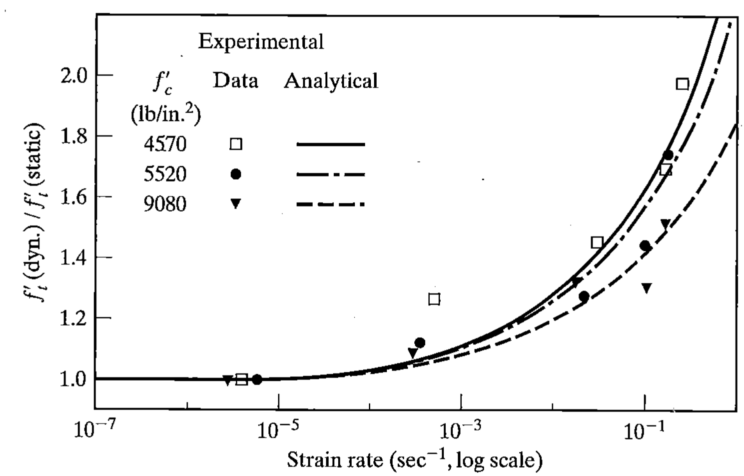


FIGURE 16.23

Effect of strain rate on the tensile strength of concrete. [Adapted from R. John and S. P. Shah in *Proceedings of SMIRT-9 Post Conference Seminar*, Impacts, Lausanne, Switzerland (1987).]

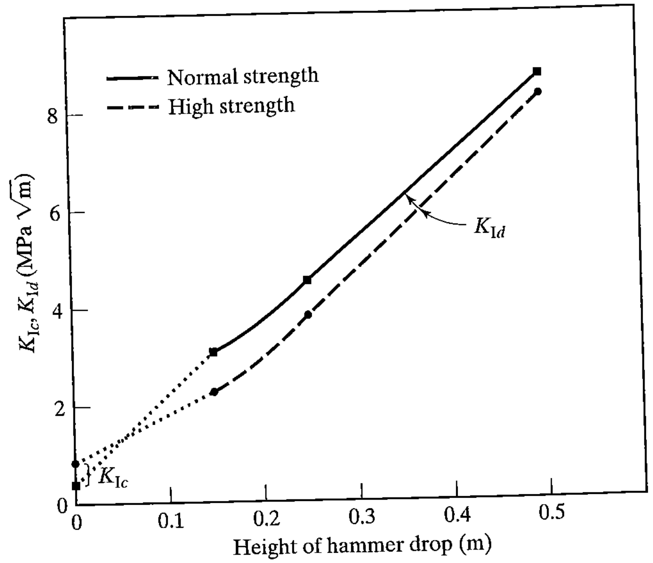


FIGURE 16.24

Fracture toughness as a function of hammer drop height for normal and high-strength concrete. [Adapted from S. Mindess, N. Banthia, and C. Yan, *Cement and Concrete Research*, Vol. 17, No. 2, pp. 231–241 (1987).]

16.4 CREEP OF CONCRETE

Although all materials undergo time-dependent deformation under load, ceramics and metals have negligible creep at room temperature. The fundamental origins for creep of concrete must be quite different, since significant volume changes occur at ambient temperatures and the presence of moisture in the material plays a major role. This discussion will concentrate on the compressive creep strains that occur under axial loading. Flexural, tensile, and torsional creep will be considered more briefly.

Relationship between Creep and Shrinkage

It is commonly stated that creep and drying shrinkage are interrelated phenomena because there are a number of similarities between the two. The strain–time curves are very similar, experimental parameters affect creep in much the same way as shrinkage, the magnitudes of the strains are the same, and they include a considerable amount of irreversibility. Like shrinkage, creep is a paste property, and the aggregate in concrete serves to act as a restraint. The origins of creep are believed to reside in the response of C–S–H to stress. Since very few creep data are available on pure pastes, it is necessary to rely on data from concretes to assess the influence of experimental variables. Table 16.1 lists the various experimental variables that can be expected to affect creep, which are the same parameters that influence drying shrinkage.

Definition of Terms

A typical creep curve is given in Figure 16.25. When a specimen is unloaded, the instantaneous recovery is approximately the same as the instantaneous strain on first application of the load, but creep recovery, although it occurs more rapidly than creep, is by no means complete: A considerable portion of the total creep is irreversible (irrecoverable). Under typical service conditions, concrete is most likely to be drying

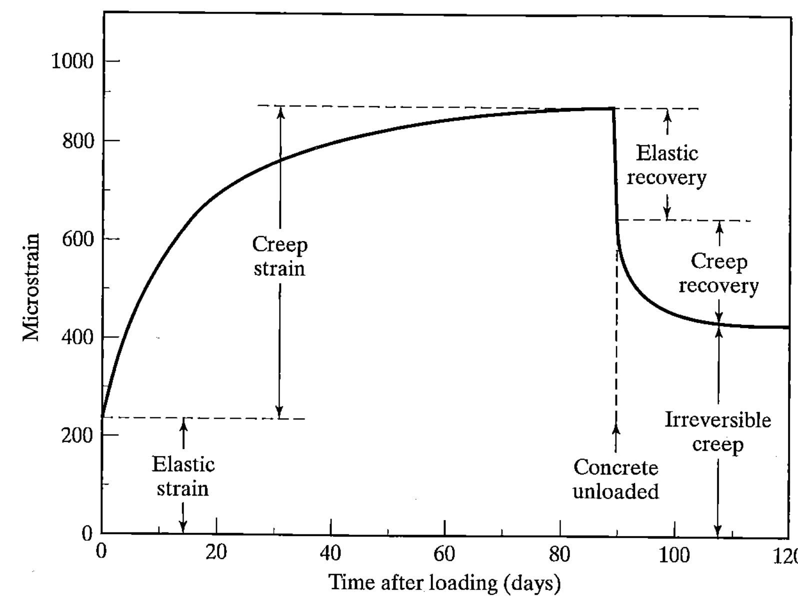


FIGURE 16.25

Typical creep curve for plain concrete.

while under load, and it has been found that under such conditions creep deformations are greater than if the concrete is dried prior to loading. Terminology has been developed to take this fact into account and is shown diagrammatically in Figure 16.26. If the free shrinkage (ε_{sh}) (determined while the specimen is unloaded, but subjected to the same drying conditions) and basic creep (ε_{bc}) (determined while specimen is loaded, but not drying) are added together, their sum is less than the total strain (ε_{tot}) determined during simultaneous loading and drying. The excess deformation is called *drying creep* (ε_{dc}). Total creep strain (ε_{cr}) is the sum of ε_{bc} and ε_{dc}. It is common practice, however, to ignore this distinction, and creep is usually considered as the deformation under load in excess of free shrinkage.

Creep is often described in terms of the *creep coefficient*

$$C = \frac{\varepsilon_{cr}}{\varepsilon_e} \tag{16.8}$$

where ε_{cr} and ε_e are, respectively, the creep and instantaneous strains under the applied load.

Factors Influencing Creep

Applied Stress It is generally assumed that creep strains are linearly related to the applied stress, up to a stress of about 50% of the ultimate strength of the concrete. However, recent work has shown that experimental data are more accurately represented by the nonlinear relationship

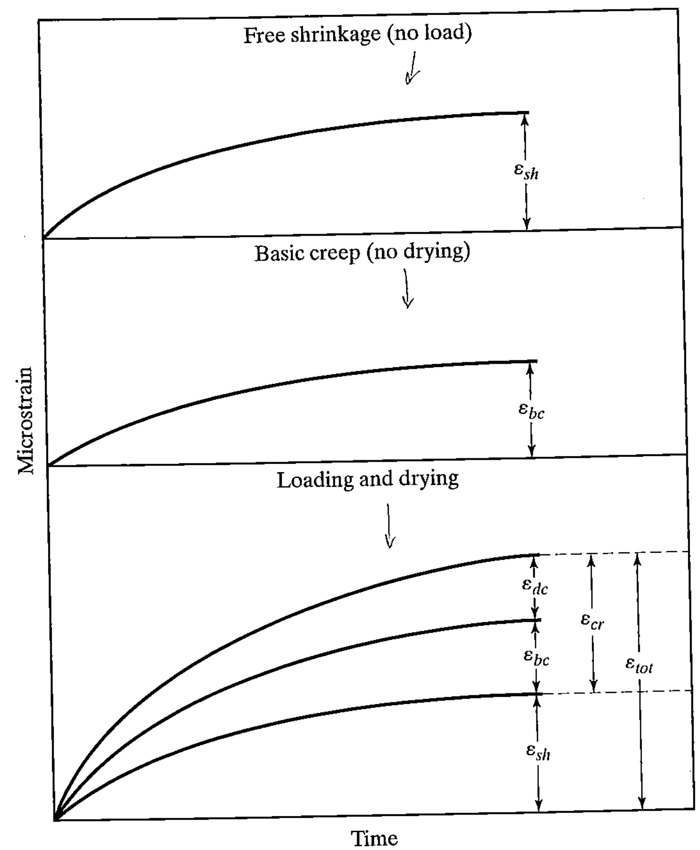


FIGURE 16.26

Creep of concrete under simultaneous loading and drying.

$$\varepsilon_{cr} = C \sinh\left(\frac{V\sigma}{RT}\right) \tag{16.9}$$

which represents a thermally activated process. (C is a constant, V the activation volume, R the gas constant, T the absolute temperature, and σ the applied stress.) When this function is represented graphically (Figure 16.27), it is found that the creep–stress relationship is nonlinear for all values of stress, but approximately linear in the stress range generally used. The nonzero intercept of the linear representation means that, even within this range, creep is not precisely proportional to applied stress. However, for practical reasons, a linear proportional relationship is often used, giving rise to the concept of *specific creep*.

$$\text{specific creep } (\phi) = \frac{\varepsilon_{cr}}{\sigma} \tag{16.10}$$

Specific creep allows creep to be compared for different concrete specimens loaded at different stress levels; a typical value is 150×10^{-6}/MPa (10^{-6}/lb/in.^{2}).

Water/Cement Ratio A study of the literature provides conflicting data concerning the effect of w/c ratio on creep. This illustrates nicely the difficulties of interpreting

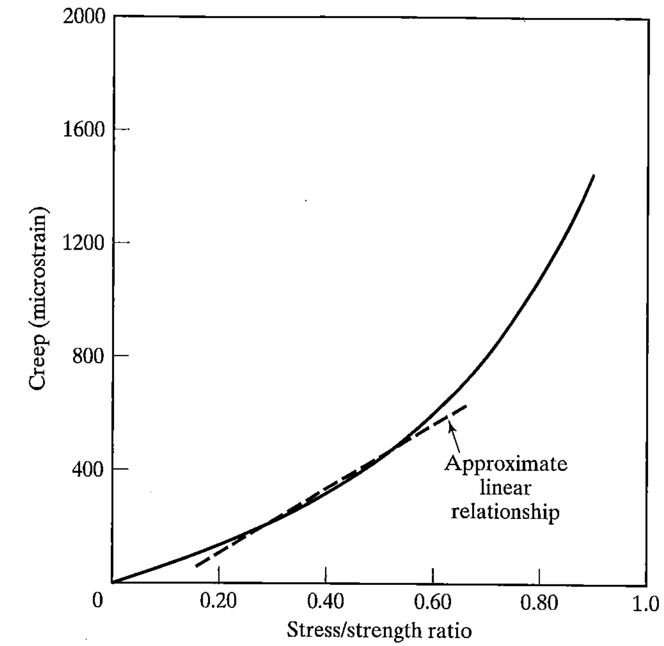


FIGURE 16.27

Creep–stress relationship for concrete.

creep data, since it is not possible to change one parameter independently of others. A change in the w/c ratio of concrete means a change in cement content and concrete strength. When allowances are made for these factors, it is found that specific creep increases with increasing w/c ratio (Figure 16.28). Thus, it might be anticipated that specific creep of concrete would be a function of its compressive strength, since that property is most strongly influenced by w/c ratio. An inverse relationship between the ultimate specific creep and the compressive strength of the concrete does indeed exist.

Curing Conditions The time of moist curing of the concrete at loading affects the magnitude of creep (Figure 16.29). This is to be expected since the degree of hydration is lower at shorter curing times and the porosity of the paste is higher. However, the age effect continues even in more mature concretes, when porosity and strength do not change markedly with time. This can be attributed to the aging effect for C–S–H, which increases its resistance to stress. Some investigators have postulated that a *maturing creep* component occurs when the concrete is loaded while it is still hydrating rapidly. An increase in the temperature of curing reduces both basic and drying creep. The amount of reduction depends on the temperature and its duration, but even quite short periods at elevated temperatures may cause significant reductions in creep. The observed reductions occur primarily in the irreversible components of creep and are similar in magnitude to the reductions in drying shrinkage under the same conditions. These observations can be rationalized by assuming that higher temperatures increase the aging process of C–S–H.

Temperature If concrete is maintained at elevated temperatures while under load, the amount of creep is increased over that of concrete held at room temperature

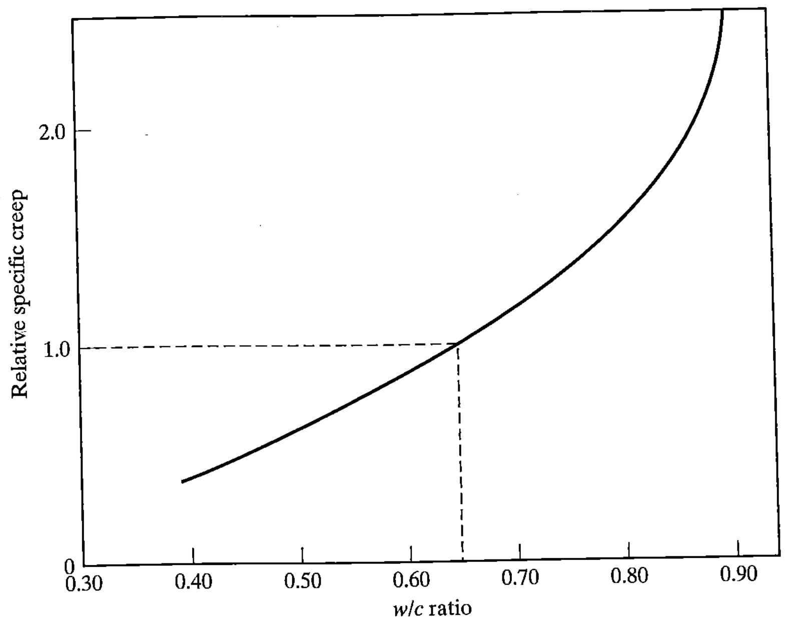


FIGURE 16.28

Effect of *w/c* ratio on creep of concrete. [Adapted from A. M. Neville, *Creep of Concrete: Plain, Reinforced and Prestressed*, North-Holland Publishing Company, Amsterdam (1970).]

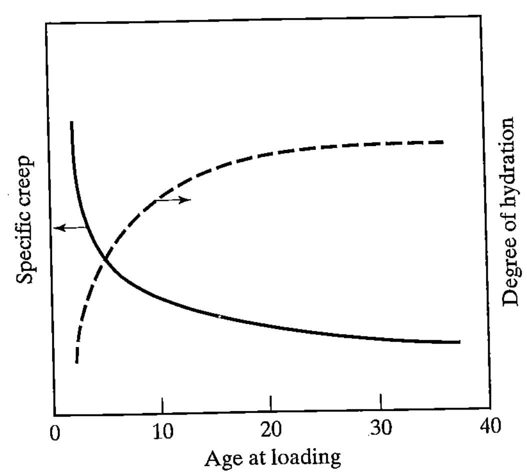


FIGURE 16.29

Effect of age on creep.

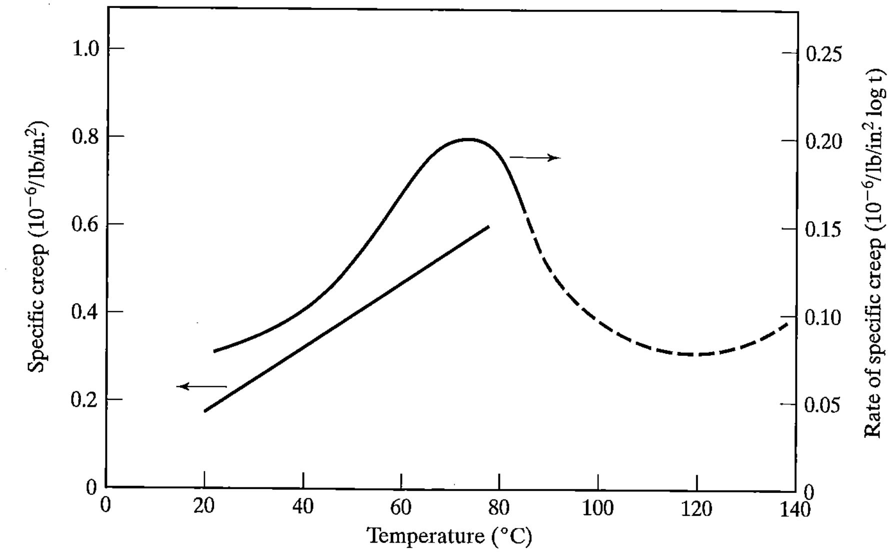


FIGURE 16.30

Effect of temperature on the rate of creep. [Based on data from H. Geymayer, in *Concrete for Nuclear Reactors*, SP-34, Vol. 1, pp. 565–589, American Concrete Institute, Farmington Hills, MI (1970).]

(Figure 16.30). Creep increases approximately linearly with temperature up to 80°C (175°F), where it is about three times the value at ambient temperature. However, above 80°C (175°F), there is some uncertainty as to the exact relationship; some investigators report a continued linear variation, but there are some reports of a maximum at 70 to 80°C (155 to 175°F). The increased creep is a result of an increase in the rate of creep (Figure 16.30), and most investigators report a maximum in the creep rate somewhere between 50 and 90°C (120 and 195°F). The reason for these conflicting data and the existence of a rate maximum is not clear, but may be the consequence of different temperature histories before loading commences. However, although creep develops faster at higher temperatures, the long-term creep should be lower due to aging. If temperature increases occur while concrete is under load, an additional creep strain component has been observed that has been called *transitional thermal creep* (Figure 16.31).

Moisture The presence of free moisture in concrete is a necessary condition for creep. By preconditioning at a lower relative humidity before applying an external load, the amount of creep is reduced. Creep is thus a function of evaporable water in the concrete [i.e., the water lost on D-drying or heating at 105°C (221°F)] and falls to zero when no evaporable water is present. The greatest decrease in evaporable water, and hence in creep, occurs on drying to 40% RH while water is being lost from capillary pores. Thus, the creep vs. water content relationship depends on the *w/c* ratio (Figure 16.32).

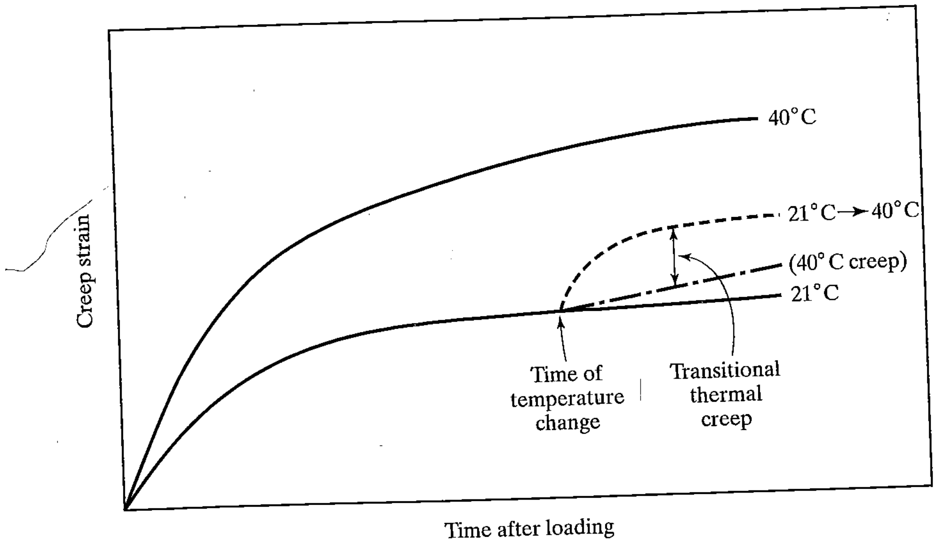


FIGURE 16.31

Effect of temperature and temperature change on creep.

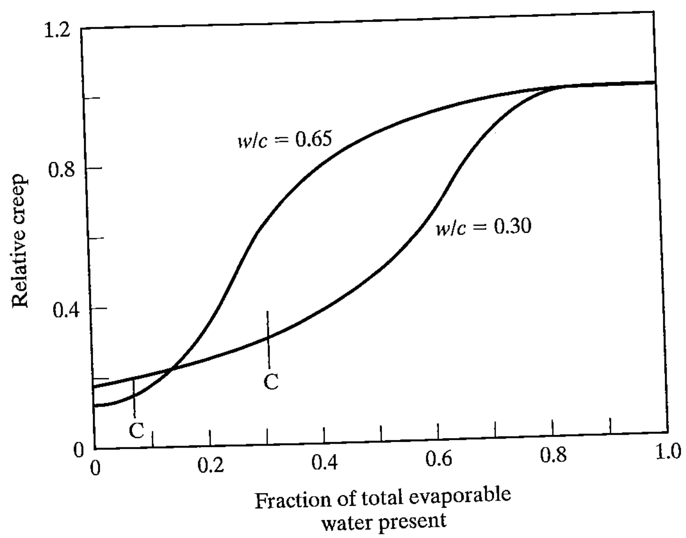


FIGURE 16.32

Creep as a function of nonevaporable water content. Normalized to concrete with all evaporable water present. (*C* represents the approximate point at which all capillary water is removed.) [Adapted from W. Ruetz in *The Structure of Concrete*, ed. A. E. Brooks and K. Newman, Cement and Concrete Association, London, pp. 146–153 (1968). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

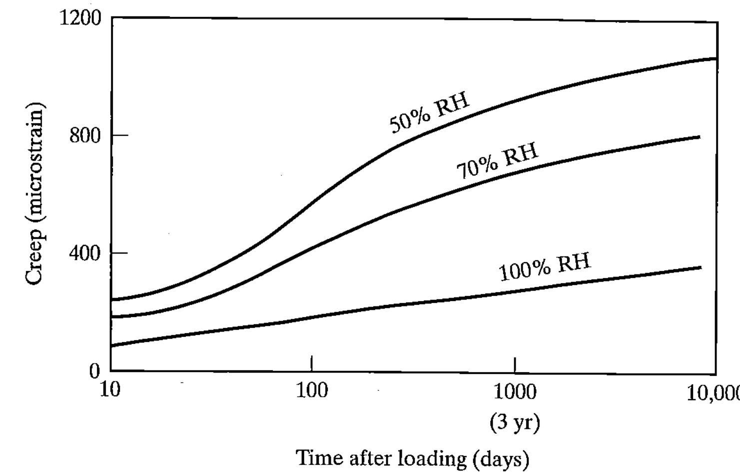


FIGURE 16.33

Creep of concrete during drying with RH change imposed on application of the load. [Adapted from G. E. Troxell, J. M. Raphael, and R. E. Davis, *Proceedings*, ASTM, Vol. 58, pp. 1101–1120 (1958).]

The amount of creep that occurs when concrete is dried as well as loaded depends on relative humidity and time, as can be seen in Figure 16.33. Drying creep strains may be considerably larger than the basic creep strain. Creep occurs during moisture change irrespective of the direction of the change. Thus, dry concrete will start to creep if it is resaturated while under load, but an appreciable rate of creep occurs only during exposure above 40% RH. Like drying creep, *wetting creep* depends on the RH change and is greatest when concrete is completely resaturated. It is believed that the disjoining pressure that causes C–S–H particles to become separated by a water film at high relative humidities (and contributes to shrinking and swelling) plays an important role in creep.

Cement Composition The relationship between the composition of a portland cement and creep is similar to that observed with shrinkage. Increasing the C_3A content or decreasing the effective C_3S content seems to increase creep. Type I cement concrete is reported to creep more than Type III. However, exact relationships are not known, as creep tests with different cements are not always comparable. Creep of pastes of the pure silicates C_3S and C_2S is less than for Type I cement pastes, and thus the presence of minor components in C–S–H and the presence of the AFm phase must have important implications for creep. There is also an "optimum gypsum content" at which creep is minimized for each cement; this tends to be higher than the optimum gypsum content at which drying shrinkage is a minimum. To a first approximation, the addition of mineral admixtures will not affect creep substantially, provided comparisons are made at the same stress-to-strength ratio.

Chemical Admixtures It is generally stated that admixtures that increase drying shrinkage also increase creep. Calcium chloride is a common example, but water-reducing and set-retarding admixtures have also been implicated, particularly lignosulfonate-based formulations. However, it has been pointed out that changes in mix design that often

accompany the use of water reducers (lower w/c ratio or lower cement content) can be used to offset possible increases in creep. Where creep is of concern, admixtures proposed for use should be tested to evaluate their influence on creep.

Influence of Aggregate The role of aggregates in creep is similar to that in shrinkage. They act as a restraint to reduce the potential deformations of the paste. Thus, the aggregate content (Figure 16.34) and modulus of elasticity (Figure 16.35) are the most important parameters affecting creep of concrete. Aggregate size, grading, and surface texture have little influence. There is an empirical relationship similar to that found for drying shrinkage [Eq. (16.5)].

Specimen Geometry When drying occurs while under load, factors such as specimen size and shape become important. Thus, the volume-to-surface ratio and specimen thickness affect total creep in much the same way as drying shrinkage is affected (Figure 16.36).

Creep Recovery

Only a relatively small proportion of the total creep strain is recoverable when concrete is unloaded; about 10 to 20% of total creep after loading for 200 days. Thus, irreversible creep dominates creep behavior, and it is of importance in predicting the behavior of concrete under variable stress. Creep recovery is strongly affected by time, temperature, and relative humidity. The proportion of irreversible creep increases with time under load. Experimental data suggest that after about the first 30 days under load, additional creep strain is largely irreversible (see Figure 16.37). The decrease in total creep that occurs when concrete is aged before loading is a result of a decrease in

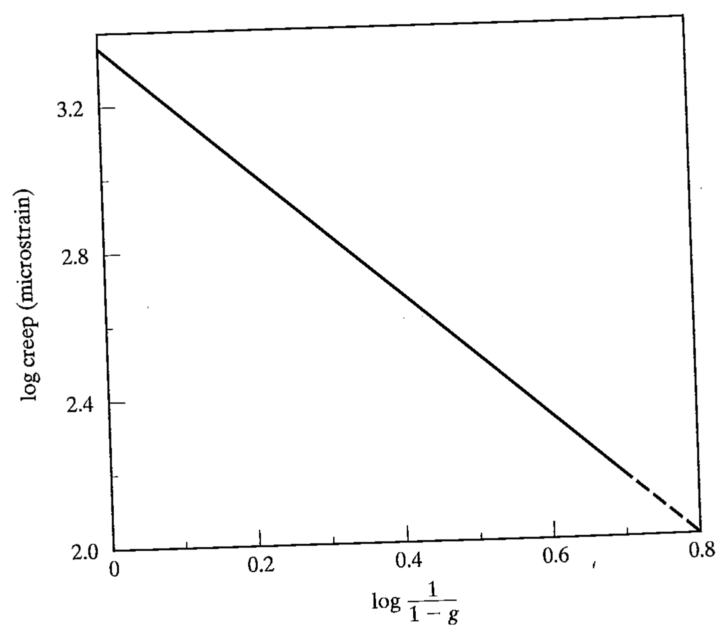


FIGURE 16.34

Effect of aggregate content by volume (g) on creep. [Adapted from A. M. Neville, *Magazine of Concrete Research*, Vol. 16, No. 46, pp. 21–30 (1964). Reproduced by permission of British Cement Association, formerly Cement and Concrete Association.]

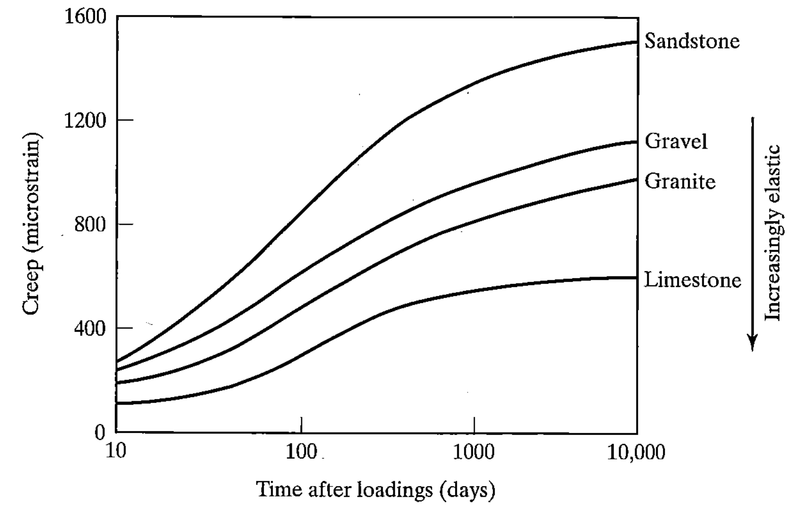


FIGURE 16.35

Effect of aggregate modulus on creep. [Adapted from G. E. Troxell, J. M. Raphael, and R. E. Davis, *Proceedings*, ASTM, Vol. 58, pp. 1101–1120 (1958).]

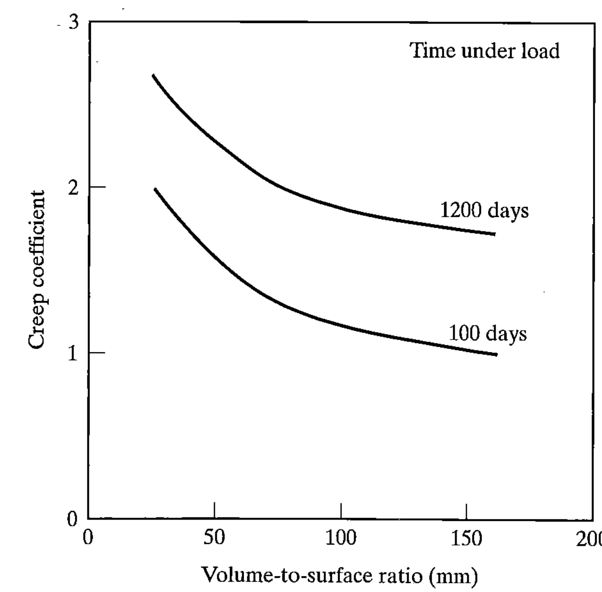


FIGURE 16.36

Effect of volume-to-surface ratio on creep of a concrete member. [Adapted from T. C. Hansen and A. H. Mattock, *Journal of the American Concrete Institute*, Vol. 63, No. 2, pp. 267–290 (1966).]

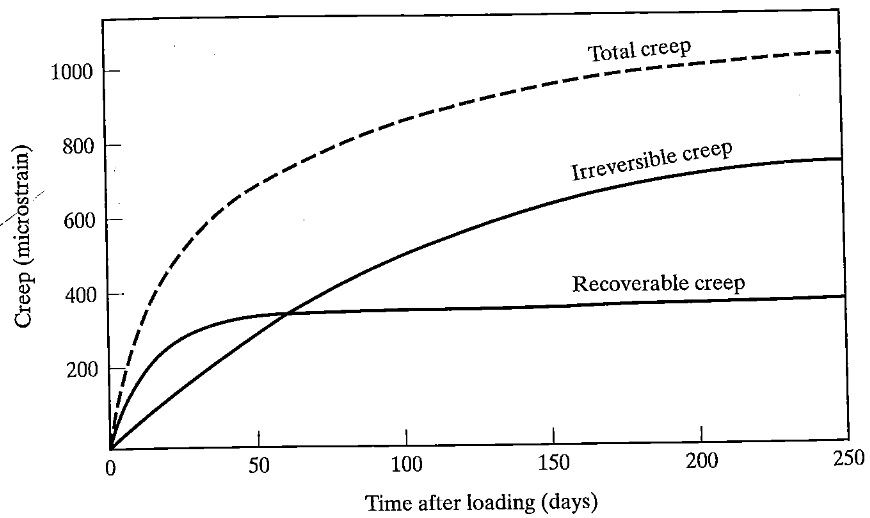


FIGURE 16.37
Influence of time on creep and creep recovery.

irreversible creep, creep recovery being little affected. Similarly, the decrease in creep that is observed when the paste is exposed to a higher temperature is the result of a decrease in irreversible creep only. Thus, it would seem that irreversible creep can be considered as being the consequence of aging processes under stress; if the C–S–H has been "aged" by prior treatment, further aging is reduced. The drying creep and transitional thermal creep are also irreversible, suggesting that they may accelerate the aging process. Increasing the applied stress also increases irreversible creep, but does not change creep recovery. Therefore, as a general rule it can be said that the external factors affect primarily the irreversible component. However, creep recovery does depend on mix parameters such as *w/c* ratio and aggregate content.

Creep under Different States of Stress

Creep in tension is of considerable interest in estimating cracking potential due to stresses imposed by moisture or thermal changes. Reduction of tensile stresses by tensile creep can minimize cracking in water-retaining structures and thin-shell roofs, where impermeability is important. However, it is difficult to measure accurately because of the low tensile strength of concrete; therefore, it is not easy to draw quantitative comparisons between creep in compression and in tension. It does appear that the initial rate of creep is higher in tension and that tensile creep is therefore greater for relatively short durations of load, although at longer times the reverse may hold. Qualitatively, creep in tension is affected by experimental variables in a way similar to creep under compressive stresses. Flexural creep is complicated by the fact that part of concrete is in compression and part in tension; creep in the tensile and compressive fibers is not necessarily the same and can be affected to different extents by drying.

Creep also occurs under dynamic loading, although it is difficult to separate out those time-dependent strains due to creep and those due to progressive microcracking under the changing stress conditions (which leads to fatigue failure). It is generally found that dynamic creep is greater than static creep compared under the same maximum stress. Creep strains appear to depend on the range of stress, the frequency of loading, and the duration of dynamic loading.

Under a uniaxial compressive stress, some lateral creep also occurs. Poisson's ratio for creep is approximately the same as for instantaneous loading, but it may appear to be less if corrections are not made for simultaneous shrinkage. It is of interest to know whether the creep strains from multiple stresses act independently of one another; that is, can creep strains be superimposed? It appears that under multiaxial loading, the creep Poisson's ratio is less than it is in uniaxial compression. Axial creep of concrete confined by lateral stress is less than for unconfined concrete after allowance is made for Poisson creep. A linear relationship is found between volumetric creep and the mean normal stress $[(\sigma_1 + \sigma_2 + \sigma_3)/3]$. Even under hydrostatic compression, there is considerable creep.

Mechanisms of Creep

Numerous theories have been advanced to explain creep of cement paste and hence of concrete. It is not possible to review all of the approaches, and the discussion will be limited to those concepts that emphasize the microstructural origins of creep.

Thermally Activated Creep This approach takes as its basic premise the assumption that the time-dependent strains are the result of thermally activated processes that can be described by rate process theory. This approach is exactly analogous to that used for other materials, and the differences between cement paste and metals, for example, lie in the different types of processes that can operate. Creep strains will originate through deformation of a microvolume of paste, called a "creep center." The creep center will undergo deformation to a lower energy configuration under the influence of energy added to the system by external sources. This deformation can only occur by going through an energy barrier (Figure 16.38) in the form of an intermediate, high-energy state.

The ability of a creep center to cross the barrier depends on the height of the energy barrier and the inputs of energy from external sources: temperature, stress (strain

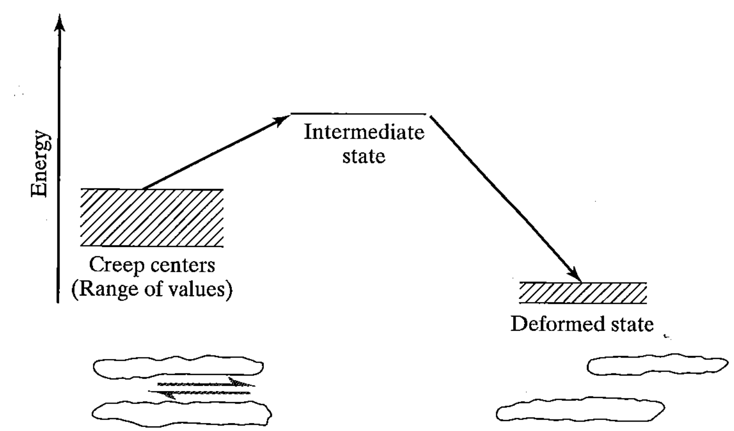


FIGURE 16.38
Schematic representation of temperature-dependent deformation of creep centers in cement paste.

energy), changes in moisture content, and changes in temperature. There is a time dependence that reflects the reduction in the number of centers as the activation barrier is overcome. Equation (16.11) is the mathematical description of this process, C being a function of the number of creep centers and their rate of annihilation as they deform to a more stable state. The size of V_a is measured in nm^3; that is, it corresponds to a section of C–S–H gel. Since an activation process describes a rate process, Eq. (16.11) should be applied to the rate of creep and can be extended to include time and changes in moisture concrete or temperature while under load.

$$\frac{d\varepsilon}{dt} = Ct^n \sinh\left(\frac{V_a}{RT}\right)\cosh\left(\frac{E\Delta X_i}{RT}\right) \tag{16.11}$$

where t is time, n is an integer, ΔX_i is a change in parameter i (moisture loss, temperature, etc.), R is the gas constant, T is temperature (K), and E is an activation energy.

Since this view of microscopic deformation automatically indicates irreversible annihilation of creep centers, how can a reversible process occur? On the removal of stress, the material can behave as if a negative stress is applied. The new stress pattern will activate a new set of creep centers, causing deformations in the opposing direction, but the total deformations will be smaller, since they are accompanied by a relaxation of the stress distribution to zero. Hence, the observed creep recovery is less than the original creep.

Role of Adsorbed Water What is the nature of the creep center, and what processes are operating? The most prevalent view involves slip between adjacent particles of C–S–H under a shear stress. The ease and extent of slip depends on the forces of attraction between the particles. If the particles are chemically bonded, no slip can occur, but if only van der Waals' interactions are operating, slip is theoretically possible. It appears that measurable slip occurs only when a sufficient thickness of water exists between the particles. The water can reduce the van der Waals' forces sufficiently to allow slippage more readily; that is, it acts as a lubricant.

Creep can also result from the diffusion of micropore water under stress, as shown schematically in Figure 16.39. This is the disjoining effect described earlier in this chapter. The thickness of the adsorbed water films that separate C–S–H particles

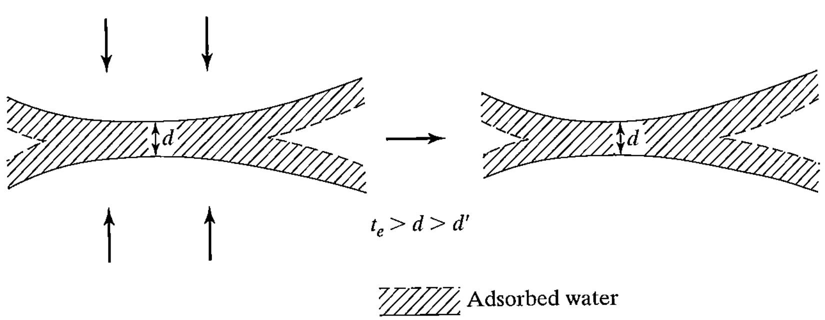


FIGURE 16.39

Schematic representation of creep due to changes in disjoining pressures.

depends on the relative humidity with which the system is in equilibrium. In a saturated paste (100% RH), the equilibrium thickness (t_e) is about five water molecules thick (about 1.3 nm). If two adjacent C–S–H particles are closer than $2t_e$, the equilibrium films cannot be attained without forcing the particles apart. If the particles are fixed, a disjoining pressure is developed. This phenomenon has been termed *hindered adsorption* and can occur only in the micropores. The equilibrium state of water in the micropores is thus determined by a combination of stress and thickness (interactions with the solid surface).

When an external stress is applied, the stress exerted on the water in the micropores is increased. To maintain equilibrium, the thickness of the adsorbed layer must be decreased to compensate for the effective increase in disjoining pressure that has been created. Thus, water diffuses from the micropores to the capillary pores where no stress exists, thereby causing a bulk deformation (see the discussion in Section 16.2). The amount of water redistributed by diffusion is a very small fraction of the total water in the cement paste, so that creep occurs even in saturated specimens without an external loss of water. The reordering of C–S–H particles can lead to an increased bonding between them, such as the formation of interlayer regions. Thus, creep can be considered a stress-induced aging leading to the elimination of micropores.

Since water movement will occur also when the external RH is lowered, this conceptual view can be considered to be a unified creep and shrinkage theory. Drying creep (or "stress-induced shrinkage") is explained by the fact that the reduction in micropore water generally occurs much more rapidly by drying than by stress. There is thus a temporary diminution of the stressed volume of water, which raises the effective stress under which diffusion occurs. It has also been suggested that diffusion of solid material as well as water can occur. For example, the calcium hydroxide can dissolve under localized stress and redeposit in unstressed regions. At higher applied stresses, internal microcracking can occur, giving rise to additional irreversible strain.

Loss of Prestress

Creep has been considered in this chapter as a problem of time-dependent strain, but it can also be manifested as a stress relaxation. Creep (and shrinkage) will thus reduce compressive stress in concrete induced by prestressing and enhance the tendency for cracking. In fact, creep frustrated early attempts at prestressing. It is thus essential to estimate the magnitude of loss of prestress due to creep and shrinkage to ensure an adequate residual prestress. It is beyond the scope of this book to consider in detail the effects of creep and shrinkage on loss of prestress. Suffice it to say that in pretensioning, the prestress is applied while the concrete is quite young and undergoes larger creep and drying shrinkage. On the other hand, post-tensioning generally involves more mature concrete, which has had an opportunity to dry. In such cases, prestress losses due to creep and shrinkage are smaller.

16.5 PREDICTION OF SHRINKAGE AND CREEP

If some allowance is not made for drying shrinkage and creep of concrete in design, unacceptable cracking or misalignment of structural elements is likely to occur. In Lake Shore Towers in Chicago, combined creep and shrinkage resulted in a shortening of the

vertical columns by 2.5 mm per floor. Many empirical equations have been developed to predict shrinkage and creep. In this section, we describe the set of constitutive equations recommended by ACI Committee 209. These equations are simple mathematical formulae to capture the effects of the different parameters that affect concrete behavior. More sophisticated equations have been proposed that can give more accurate predictions using modern computers, and these can be used when it is necessary to make more precise predictions. The advantage of the ACI equations is that they are very simple and yet provide a level of accuracy that is adequate for most structures.

Drying Shrinkage

Basic Equations The basic equations describing the shape of the shrinkage–time curve is given by the hyperbolic relationship

$$(\varepsilon_{sh})_t = \frac{t}{\phi + t}(\varepsilon_{sh})_u \tag{16.12}$$

where $(\varepsilon_{sh})_t$ is the shrinkage after t days of drying, $(\varepsilon_{sh})_u$ is the ultimate drying shrinkage, and ϕ is an empirical constant. Equation (16.12) applies to concrete moist-cured for seven days and then dried at 40% RH. The value of ϕ lies between 20 and 120 days; but for regular concrete cured for seven days at room temperature, a value of 35 is used. Similarly $(\varepsilon_{sh})_u$ lies between 415×10^{-6} and 1070×10^{-6}. Determining ultimate shrinkage values from short-term tests can be prone to error. If a measured value is not available, 780×10^{-6} is recommended. Thus, the default form of Eq. (16.12) is

$$(\varepsilon_{sh})_t = \frac{t}{35 + t}780 \times 10^{-6} \tag{16.13}$$

Correction Factors Equation (16.12) can be modified to account for other factors that affect shrinkage. There are several alternate methods that can be used, but the simplest involves (1) modifying the value of ϕ and (2) modifying the value of $(\varepsilon_{sh})_u$.

Modifying ϕ. Reduction of shrinkage by steam curing for one to three days (precast concrete) is handled by making $\phi = 55$. The effect of specimen shape and size can be handled by calculating ϕ according to

$$\phi = 26\exp[0.0142(V/S)] \tag{16.14}$$

where V/S is the volume-to-surface ratio.

Modifying $(\varepsilon_{sh})_u$. In this approach, the value of $(\varepsilon_{sh})_u$ is modified by multiplying by a correction factor γ_c, which is the product of different parameters

$$\gamma_c = \gamma_{rh}\gamma_{mc}\gamma_{cp} \tag{16.15}$$

γ_{rh} is an important correction for the RH of exposure and is given by

$$40\% \le \text{RH} \le 80\%: \gamma_{rh} = 1.40 - 0.01(\text{RH}) \tag{16.16a}$$

$$\text{RH} > 80\%: \gamma_{rh} = 3.00 - 0.03(\text{RH}) \tag{16.16b}$$

γ_{mc} is the adjustment for different moist-curing periods other than 7 days, ranging from 1.2 for curing for 1 day prior to drying to 0.75 for 90 days moist curing. γ_{mc} varies linearly with the semilog of curing time. γ_{cp} is a factor to account for changes in mix design (cement content, air content, fine aggregate content, etc.), but can be neglected in most applications.

Creep

Basic Equations The basic equation describing the creep–time relationship is given by

$$C_t = \frac{t^{0.6}}{D + t^{0.6}} C_u \tag{16.17}$$

where C_t is the creep coefficient after t days under load, C_u is the ultimate creep, and D is a constant. Equation (16.17) applies to concrete moist cured for seven days before loading in a 40% RH environment. D can range from 6 to 30 days, but for normal concrete moist cured for 7 days before loading, a value of 10 is used. This value is also used for steam-cured concretes. Estimating C_u can be a problem; the value can range from 1.30 to 4.15. A default value of 2.35 is recommended, so that Eq. (16.17) becomes

$$C_t = \frac{t^{0.6}}{10 + t^{0.6}}2.35 \tag{16.18}$$

Correction Factors Allowing for changes in creep behavior under other conditions is handled by applying correction factors to C_u, as described in Eq. (16.15). The following are the most important corrections: Relative humidity other than 40% RH is found with

$$\gamma_{rh} = 1.27 - 0.0067(\text{RH}) \tag{16.19}$$

The effect of loading age is given by

$$\text{Moist cured concrete:} \quad \gamma_{la} = 1.25\,t^{-0.118} \tag{16.20a}$$

$$\text{Steam-cured concrete:} \quad \gamma_{la} = 1.13\,t^{-0.094} \tag{16.20b}$$

The effects of changes in mix design are small and usually can be neglected. Corrections for changes in size and shape of a concrete component are handled in the same way as for drying shrinkage.

Limitations A major limitation of the ACI approach is the omission of the effects of changes in temperature during loading. Nor is any attempt made to separate out reversible and irreversible components. The predictions could be modified to include these effects. The ACI approach is too simplistic to predict the complex interplay of changes in loading (e.g., cyclic loading), relative humidity, and temperature on drying shrinkage and creep. Attempts have been made to capture these effects with the use of powerful computing routines in cases where in-service performance needs to be accurately predicted (e.g., nuclear containment structures).

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PROBLEMS

16.1. What effect does increasing the rate of loading have on (a) concrete strength; (b) the strain corresponding to peak stress?

16.2. Why does concrete respond as described in the answer to Problem 16.1?

16.3. What effect does moisture content have on the strength and stiffness response of concrete at increasing strain rates?

16.4. A concrete column with a 28-day cylinder strength of 4500 psi (31 MPa) is calculated to bear an applied compressive load equivalent to 0.3 of its strength. If the creep coefficient after one year is found to be 1.85, calculate the strain in the column. What is its specific creep?

16.5. A concrete slab is exposed to drying at 75% RH seven days after casting. Compute the drying shrinkage strain after (a) 60 days; (b) 180 days.

16.6. Precast concrete panels (12 ft × 8 ft) are steam cured for 24 hours and then stored in the precast yard for 30 days at a mean RH of 85% prior to being installed on a building. What will be the actual dimensions of the panel at the time of installation? What will be the additional potential free shrinkage over the first year in service if the mean RH of exposure is 70% RH.

16.7. Steam-cured precast beams are prestressed after 24 hours when the compressive strength reaches 3500 psi. The level of prestress is 1000 psi. Determine the potential free strain that will occur over the first year if the beams are exposed to 70% RH.

CHAPTER 17

Other Properties of Concrete

17.1 THERMAL PROPERTIES

The properties of concrete, like those of other materials, are affected by changes of temperature, as discussed in the appropriate chapters. This section is concerned with the thermal properties of concrete—thermal expansion, thermal conductivity, and specific heat—as well as a general discussion of the response of concrete when exposed to extreme temperatures. These are of special concern in the design of structures where the concrete may be (1) exposed to thermal stresses or sustained high temperatures, (2) used for thermal insulation purposes, (3) used for fire protection, or (4) used for cryogenic applications. These categories cover a wide variety of practical applications. The thermal properties are more complex than they are for most materials, because not only is concrete a composite material whose components have different thermal properties (see Table 17.1), but its properties depend on moisture content and porosity.

Thermal Expansion

Thermal expansion is an important factor in all types of structures where differential heating may occur, either from environmental effects, such as the solar heating of pavements and bridge decks, or from service conditions, as in nuclear-reactor pressure vessels or furnace installations. Failure to allow for thermal expansion, or for thermal stresses resulting from differential expansion, will cause failure. The differential expansion that can occur between cement paste and aggregate will give rise to high internal stresses, which may be critical in cases where there are large temperature changes.

Cement Paste The coefficient of thermal expansion, in the range −10 to 100°C (14 to 212°F), is not a unique value, but depends on the moisture content of the paste (Figure 17.1), the *w/c* ratio, and the age of the paste. There is an unusual moisture dependency, in which the coefficient increases considerably at intermediate relative humidities. This has been explained by considering that internal rearrangement of water takes place between capillary pores and gel pores without a change in the total water content of

TABLE 17.1 Thermal Properties of Concrete Constituents

	Thermal Conductivity W/m·K (Btu/ft·h·°F)		Specific Heat, J/kg·°C (Btu/lb·°F)	Coefficient of Thermal Expansio, 10^{-6}/°C (10^{-6}/°F)	
Aggregate					
Granite	3.1	(1.8)	800 (0.19)	7–9	(4–5)
Basalt	1.4	(0.8)	840 (0.20)	6–8	(3.3–4.4)
Limestone	3.1	(1.8)	—	6	(3.3)
Dolomite	3.6	(2.1)	—	7–10	(4–5.5)
Sandstone	3.9	(2.3)	—	11–12	(6.1–6.7)
Quartzite	4.3	(2.5)	—	11–13	(6.1–7.2)
Marble	2.7	(1.6)	—	4–7	(2.2–4)
Cement paste[a]					
w/c = 0.4	1.3	(0.75)	—	18–20	(10–11)
w/c = 0.5	1.2	(0.7)	—	18–20	(10–11)
w/c = 0.6	1.0	(0.6)	1600 (0.38)	18–20	(10–11)
Concrete	1.5–3.5	(0.9–2.0)	840–1170 (0.2–0.28)	7.4–13	(4.1–7.3)
Water	0.5	(0.3)	4200 (1.0)	—	—
Air	0.03	(0.02)	1050 (0.25)	—	—
Steel	120	(70)	460 (0.11)	11–12	(6.1–6.7)

[a] Saturated condition.

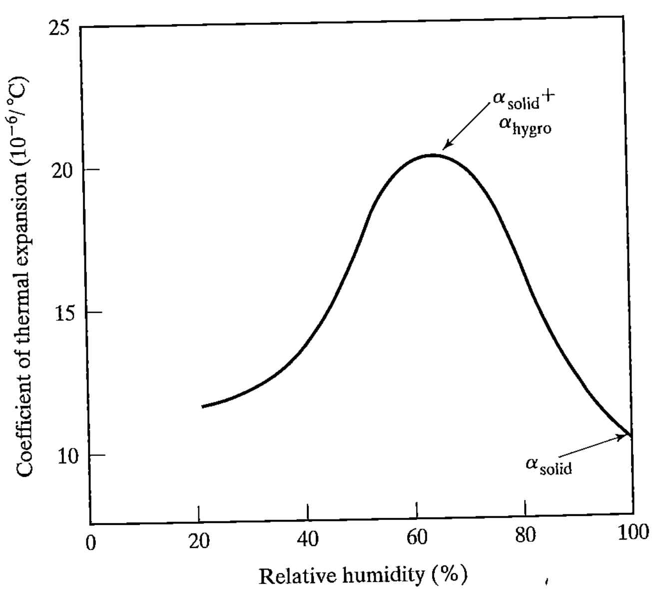


FIGURE 17.1

Variation of the coefficient of thermal expansion with moisture content of cement paste (equilibrated prior to measurement). [Adapted from N. G. Zoldners, in *Temperature and Concrete*, SP-25, American Concrete Institute, Detroit, MI, pp. 1–31 (1971).]

the paste. This can be called *hygrothermal expansion,* and it is dependent on w/c ratio and age, because these factors determine the porosity characteristics of the paste. The following relationship holds:

$$\alpha_{\text{actual}} = \alpha_{\text{solid}} + \alpha_{\text{hygro}} \qquad (17.1)$$

where, α_{actual} is the measured coefficient of thermal expansion; α_{solid} is that measured in the absence of hygrothermal change, depending (as with other materials) on kinetic molecular movement; and α_{hygro} is the coefficient of hygrothermal expansion. It is believed that α_{solid} is independent of paste properties, averaging about 10×10^{-6}/°C (5.5×10^{-6}/°F).

If a moist paste is heated, loss of moisture should be accompanied by shrinkage. The amount of shrinkage that can occur will depend on the duration of heating, the permeability of the paste, and the thickness of the specimen. Below 100°C (212°F), shrinkage due to loss of moisture will be compensated for by an increase in the thermal coefficient at lower water contents, and a net expansion may occur (Figure 17.2). But a net contraction will be observed at higher temperatures until all moisture has been lost from the paste. Much greater shrinkage is observed for a given amount of moisture loss above 100°C than for the same loss below 100°C, because structural breakdown of the hydration products is occurring.

Aggregates It can be seen from Table 17.1 that the coefficient of thermal expansion for common rocks varies with the mineralogical composition. Within each type, a range of values occurs that depends on chemical composition, structure, and porosity; rocks are themselves composite materials. An average value for most rocks is in the vicinity of 6 to 8×10^{-6}/°C (3 to 4×10^{-6}/°F), which is less than α_{solid} for cement paste and

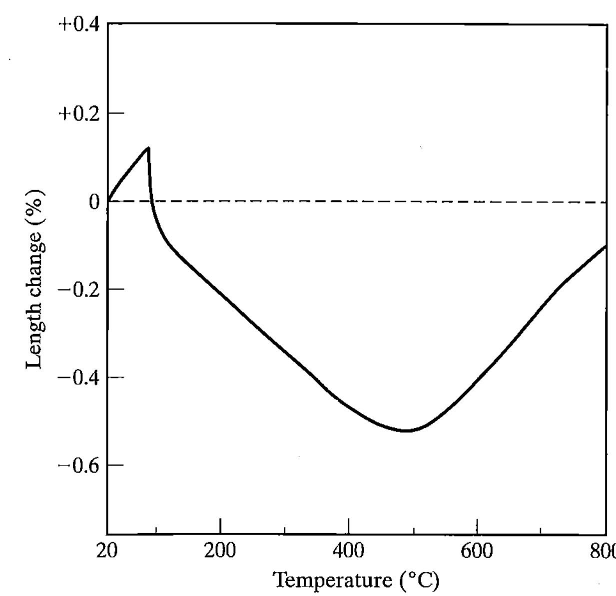


FIGURE 17.2

Length change of cement pastes heated to elevated temperatures. [Adapted from N. G. Zoldners, in *Temperature and Concrete*, SP-25, American Concrete Institute, Detroit, MI, pp. 1–31 (1971).]

considerably less than α_{actual} (Figure 17.1). Quartz has the highest coefficient of thermal expansion (12×10^{-6}/°C or 7×10^{-6}/°F) of any common mineral, and the coefficients of various rocks are related to their quartz content. Rocks with a high quartz content (quartzites, sandstones) have coefficients similar to quartz; those containing no quartz (limestones, marbles) have coefficients around 5×10^{-6}/°C (3×10^{-6}/°F).

The coefficient of thermal expansion of rocks also depends on moisture content, but to a much lesser degree than cement pastes. Air-dry rocks may expand about 10% more than water-saturated rocks. Temperature has a much greater effect on the thermal coefficient (Table 17.2), which increases markedly, but nonlinearly, with temperature. In contrast, the thermal coefficient of cement paste does not change appreciably with temperature. However, at high temperatures other changes may also take place. Rocks may lose integrity due to chemical decomposition (limestone, basalt) or phase changes (quartzite). Anorthositic rocks show the best thermal stability, being relatively stable up to 1000°C (1830°F).

Concrete The coefficient of thermal expansion will be a variable quantity depending on the mix design and the type of aggregate used. Since aggregates make up the bulk of concrete, their properties will largely determine the concrete properties. In a mortar, the coefficient of thermal expansion will be a linear function of the volume of sand and cement (Figure 17.3a). The coefficient of thermal expansion of concrete can be estimated from the volumes of mortar and coarse aggregate (Figure 17.3b). Because cement paste has a high thermal expansion, the coefficient will also depend on the cement content, although the variation over the normal range of cement contents may not be as great as changing the type of aggregate.

Thermal Conductivity

Thermal conductivity can be defined as the ratio of heat flux to temperature gradient. The coefficient of thermal conductivity, k, represents the uniform flow of heat through a unit thickness of material between two faces of unit area that are subjected to a unit temperature difference. It is an important property in the design of concrete installations exposed to heat since it determines the rate of penetration of heat into the concrete and hence the magnitude of temperature gradients and thermal stresses. The thermal conductivity of concrete will be determined by the values for cement paste

TABLE 17.2 Effect of Temperature on the Average Coefficient of Thermal Expansion of Some Aggregates (10^{-6}/°C)[a]

Temperature Range (°C)	*Rock Type*			
	Sandstone	*Limestone*	*Granite*	*Anorthosite*
20–100	10	3	4	4
100–300	15	9	14	9
300–500	22	17	26	10
500–700	25	33	48	13

[a] $°C \times \frac{9}{5} + 32 = °F$; $10^{-6}/°C \times \frac{5}{9} = 10^{-6}/°F$.

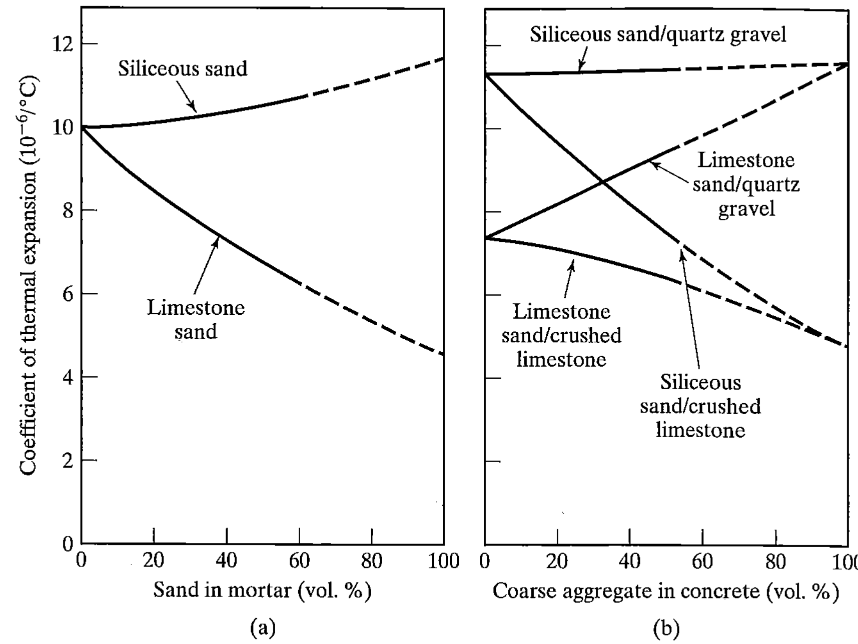


FIGURE 17.3

Effect of aggregate content on the thermal expansion of (a) mortar; (b) concrete. (Dotted lines are an extrapolation of experimental data.) [Adapted from N. G. Zoldners, in *Temperature and Concrete*, SP-25, American Concrete Institute, Detroit, MI, pp. 1–31 (1971).]

and aggregate and their relative proportions in the mix. The thermal conductivities of paste and aggregate are strongly influenced by porosity and moisture contents, and so, too, is that of concrete. This is not surprising, since the medium contained in the pore system also contributes to heat flow. Reference to Table 17.1 shows the thermal conductivity of water and air to be markedly different and in both cases substantially lower than the value for a solid material.

Neat cement pastes have thermal conductivities in the range 1.0 to 1.5 W/m · K, the value depending on the *w/c* ratio (Table 17.1). The degree of saturation has a greater influence than the total porosity; however, in strongly dried pastes, where all pore water has been removed, the value of the coefficient drops to about half that of the saturated paste. The thermal conductivity of most rocks is approximately 3.0 W/m · K (Table 17.1). Notable exceptions are quartz and dolomite. The moisture content may also have a significant effect on the thermal conductivity of rocks.

Thermal conductivity is related to the density of concrete (Figure 17.4); lightweight concretes that have high porosities have very low thermal conductivities, because of the large volume of air voids. This accounts for their good thermal insulating properties; it is immaterial whether the porosity is contained within the aggregate (lightweight aggregates) or within the paste (foamed or aerated concretes). Since the

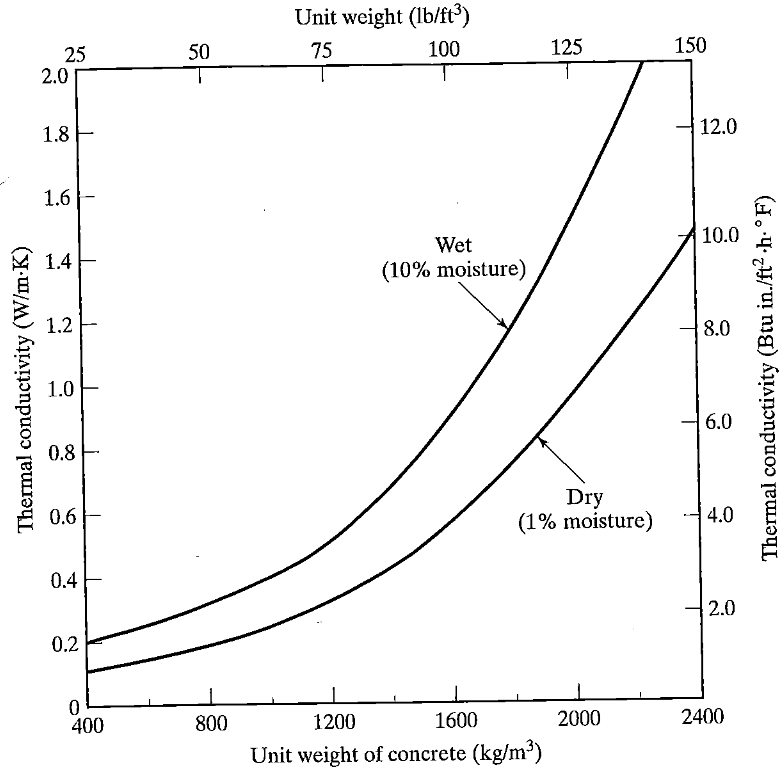


FIGURE 17.4

Thermal conductivity as a function of density and moisture content. [Based on data from A. Short and W. Kinniburgh, *Lightweight Concrete*, 3rd ed., Applied Science Publishers, Ltd., London (1978).]

thermal conductivity of both paste and aggregate depends on the degree of saturation, the same is true for concrete (Figure 17.4). The actual effect that a change of moisture content has on thermal conductivity depends on the initial degree of saturation of the concrete. The thermal conductivities of paste, aggregate, and concrete are independent of temperature within the normal climatic range. Above 100°C, however, thermal conductivity decreases linearly with temperature (Figure 17.5). This has been attributed to changes in moisture content on drying.

Specific Heat

Specific heat, or heat capacity, is little affected by the type of aggregate, since the specific heats of rocks do not change much with mineralogical type. The specific heat of cement paste, however, is strongly dependent on porosity (*w/c* ratio), water content, and temperature (Table 17.3). The specific heat of concrete also depends on these factors. The common range of values for concrete is 800 to 1200 J/kg · °C.

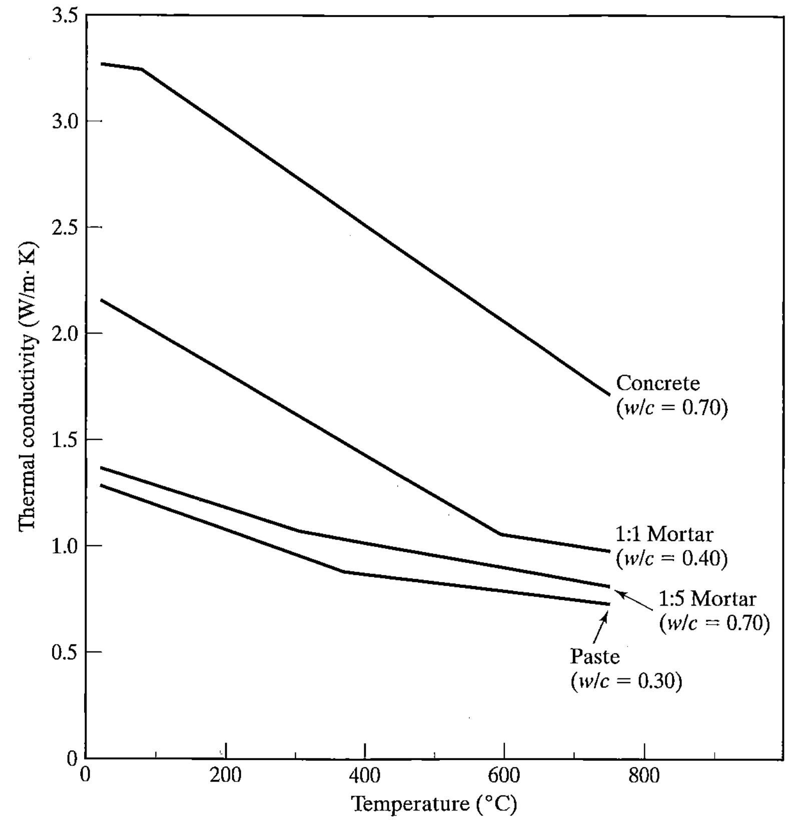


FIGURE 17.5

Variation of thermal conductivity with temperature. [Based on data from T. Harada et al., in *Concrete for Nuclear Reactors*, Vol. I, American Concrete Institute, Detroit, MI, pp. 377–406 (1972).]

TABLE 17.3 Specific Heats of Pastes, Concretes, and Mortars

Material	*a/c* *Ratio*	*w/c* *Ratio*	*Temperature* (°C)	*Specific Heat* J/kg · °C	(Btu/lb · °F)
Neat paste	—	0.25	21	1140	(0.27)
	—		65	1680	(0.40)
	—	0.60	21	1600	(0.38)
	—		65	2460	(0.58)
Mortar	1:1	—	21	1720	(0.41)
	1:2	—	21	1180	(0.28)
	1:6	—	21	1100	(0.26)
Concrete	—	—	—	800–200	(0.20–0.28)

Thermal Diffusivity

The thermal diffusivity measures the rate at which temperature changes take place in the concrete and is a function of both thermal conductivity and specific heat.

$$D = \frac{k}{c\rho} \tag{17.2}$$

where D is the diffusivity constant, k the thermal conductivity, c the specific heat, and ρ the density. Diffusivity is often measured experimentally for the determination of thermal conductivity. Obviously, factors that affect conductivity and specific heat will also affect diffusivity. Typical values for concrete range from 0.002 to 0.007 m^2/h (0.02 to 0.08 ft^2/h).

Exposure to High Temperatures

From the foregoing discussions, it can be concluded that within the normal environmental temperature range, the thermal properties of a concrete can be considered to be constant, *provided that there is no change in moisture content.* However, at elevated temperatures these properties change because of changes in the moisture content of the concrete components and because of progressive deterioration of the paste and in some cases of the aggregate. These processes depend on the conditions of exposure: the rate of temperature rise, the maximum temperature, and the time at elevated temperatures. The response of concrete will also depend on its initial properties and those of its constituents. Therefore, prediction of concrete behavior at elevated temperatures is a difficult problem.

Strength Unless large temperature differentials are allowed to develop (as in rapid heating), the compressive strength of concrete at elevated temperatures is usually maintained up to about 300°C (570°F). However, above this temperature, significant decreases can be anticipated (Figure 17.6). The magnitude of the decreases depends on the nature of the aggregate and the initial moisture content of the specimen. The changes in both strength and modulus have been attributed to a combination of decomposition of the hydrated pastes, deterioration of the aggregates, and thermal incompatibilities between paste and aggregate leading to stress concentrations and microcracking. The effect on flexural strengths is more marked, and this would be anticipated since flexural strength is more sensitive to the internal microcracking that would be expected to occur. When concretes are cooled back to room temperature before testing, the strength is less than that found if the concrete is tested while hot. This may be due in part to the imposition of additional thermal stresses and also to the rehydration of those hydration products which have been partially dehydrated, thereby causing in situ expansions.

If concretes are heated in a sealed condition to prevent loss of moisture, or when loss of moisture is slow, the cement paste is effectively subjected to high-temperature autoclaving conditions. Over short periods of time, additional reaction of the cement may result in a slight increase in strength, but eventually considerable loss of strength is observed due to the formation of crystalline calcium silicate hydrates, which is accompanied by an increase in porosity.

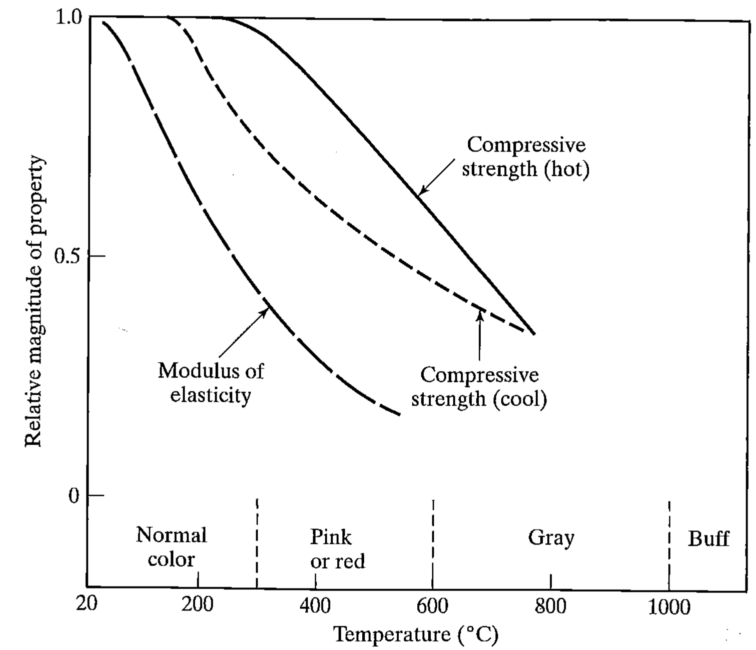


FIGURE 17.6

Effect of heating on strength and modulus of concrete.

Deformations The modulus of elasticity also decreases with temperature (Figure 17.6), and the change is more marked than in the case of compressive strength. The decrease is greater than would be expected from a change in bonding energies, and it is believed that internal microcracking at the paste–aggregate interface contributes to the change in modulus.

As mentioned earlier, drying shrinkage occurs when concrete is heated to elevated temperatures due to additional loss of moisture from the paste (Figure 17.2). The drying shrinkage at 100°C (212°F) is four to five times higher than that at 21°C (70°F) under comparable conditions. Shrinkage continues to increase at higher temperatures (Figure 17.7) as structural decomposition of the hydration products continues, and much of this is irreversible. The rate of shrinkage depends on the rate of moisture loss from the concrete and thus depends on such factors as water content, *w/c* ratio, aggregate content, specimen geometry, and drying conditions at the surface.

Creep will also increase with increasing temperature. Between 50 and 140°C (120 and 285°F), there are conflicting data concerning the magnitude of specific creep and the rate of creep (see Chapter 16). At higher temperatures, the rate of creep can be expected to increase, but the amount of creep should depend on the extent of moisture loss from the concrete.

Fire Resistance Compared to structural steel, concrete has excellent fire-resistant properties and is often used to protect steel from the effects of fire. Concrete has a lower thermal conductivity and a higher specific heat than metals; indeed, its properties

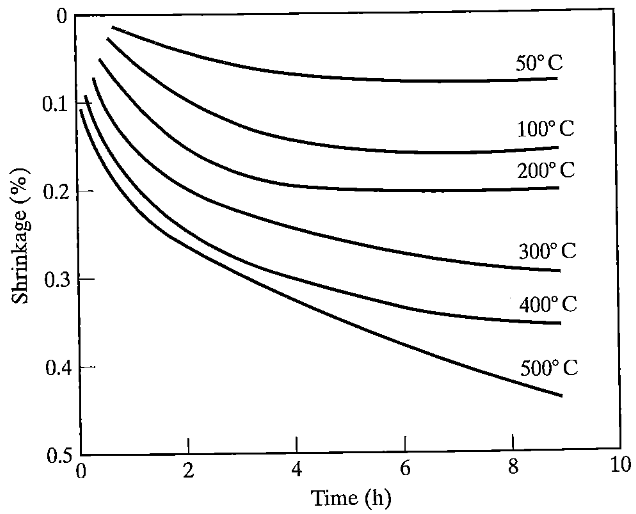


FIGURE 17.7

Effect of temperature on magnitude and rate of shrinkage. [Adapted from N. G. Zoldners, in *Temperature and Concrete*, SP-25, American Concrete Institute, Detroit, MI, pp. 1–31 (1971).]

are typical of ceramic materials. However, as we have seen, concrete is damaged by exposure to high temperatures and will suffer loss of strength, cracking, and spalling. The advantages of concrete in exposure to fire are

1. A low rate of penetration.
2. Retention of strength if exposure is not too long.
3. Absence of toxic fumes.

The low rate of heat penetration is a consequence of the low thermal conductivity; hence, lightweight concrete has better fire resistance than ordinary concrete, but other factors can also slow down the rise of temperature in concrete. Water has a high specific heat, and considerable amounts of heat are consumed in the evaporation of water. Therefore, a moist concrete may heat more slowly, although concomitant shrinkage may lead to surface spalling and cracking. Under certain circumstances, aggregates may improve fire resistance. For example, dolomitic aggregates lose carbon dioxode when heated to about 600°C; this process consumes heat and also forms a layer of porous calcined material that can insulate the interior of thick cross sections of concrete.

Concretes made with limestone or siliceous aggregates show a color change on heating (Figure 17.6). This color change persists on cooling and so can be a useful guide to the extent of fire damage. Concrete that has passed beyond the pink stage is likely to be severely damaged.

Methods of Test

Many different tests are available for the determination of thermal properties of materials. Two tests (ASTM C 177 and C 135) are available for the measurement of thermal conductivity of insulating concretes. The U.S. Corps of Engineers has standard tests for specific heat and thermal diffusivity in its *Handbook for Cement and Concrete*. ASTM

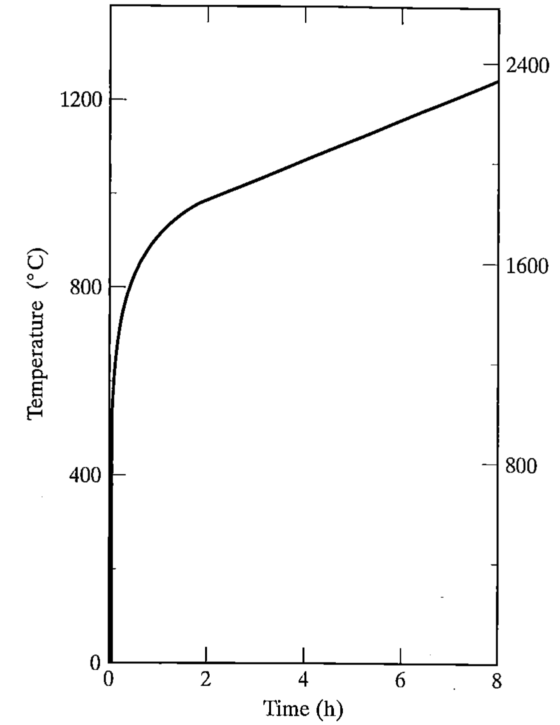


FIGURE 17.8

Standard time–temperature curve specified by ASTM E 119. (Copyright ASTM INTERNATIONAL. Reprinted with permission.)

E 119 is the only established test for fire resistance for concrete, although other special tests may well be used for particular applications. ASTM E 119 requires a fixed time–temperature curve (Figure 17.8) to be developed by the furnace used to heat the sample. The test can be applied to various specimens representing structural or nonstructural elements of a building. The success of a test can depend on any or all of the following criteria: ability to sustain the applied load, prevention of the passage of flame or hot gases through the sample, or limitation of a specified peak temperature on the face not exposed to heat. Fire resistance is expressed in terms of the time during which the specimen meets the criteria under the conditions of test. Fire-resistance testing is a specialized undertaking, and only two or three laboratories in North America are equipped for the job. Special tests may be used depending on the particular requirements.

Cryogenic Applications

Concrete is also used at very low temperatures, such as in tanks for the storage of liquefied propane gas. If the concrete is cooled slowly and not exposed to thermal cycling, it will perform satisfactorily. The freezing of capillary moisture will allow concrete that has been previously well cured to attain much higher strengths (two to three times higher) than the same concrete tested at room temperature (Figure 17.9). Dry concrete will be only slightly stronger when frozen. The modulus of elasticity is also increased at lower temperatures, while the coefficient of thermal expansion is decreased somewhat.

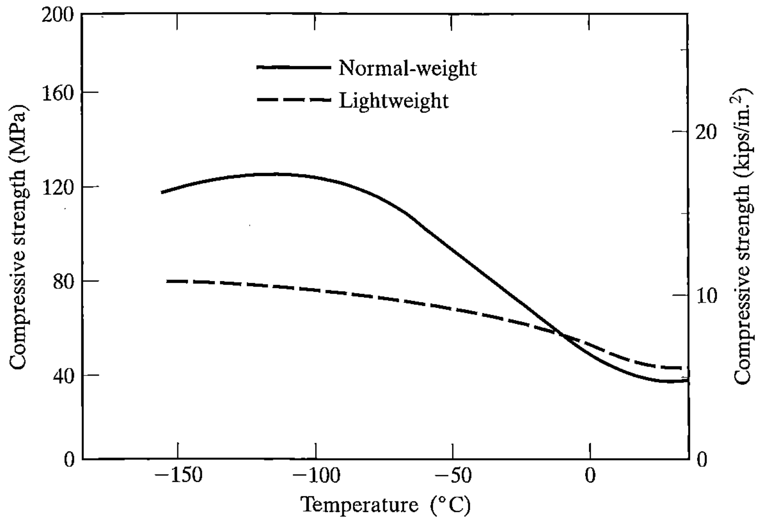


FIGURE 17.9

Effect of cryogenic temperatures on concrete strength. [Adapted from H. Woods, *Durability of Concrete Construction*, Monograph No. 4, American Concrete Institute, Detroit, MI, (1968).]

Concrete will undergo very large and irreversible losses in strength after only one or two cycles of freezing to very low temperatures (below about −70°C) and then thawing at ordinary room temperature. However, there is some evidence that if the concrete is dried before the temperature cycling begins, losses in strength may be relatively small.

17.2 RESISTANCE TO WEAR

In certain applications, severe wearing of concrete surfaces may lead to service problems. Three distinct types of wear have been distinguished:

1. *Abrasion.* Wearing by repeated rubbing or frictional processes (attrition). This term is used in connection with traffic wear on pavements and industrial floors.
2. *Erosion.* Wearing by the abrasive action of fluids and suspended solids. Erosion is a special case of abrasion and occurs in water-supply installations: canals, conduits, pipes, and spillways.
3. *Cavitation.* Impact damage caused when a high-velocity liquid flow is disturbed. It will occur at spillways and sluiceways in dams and irrigation installations.

Abrasion Resistance

Cement paste itself does not have good abrasion resistance, and the performance of concrete depends a great deal on the hardness of the aggregates used. Especially hard aggregates such as emery or iron shot have been used in abrasion-resistant toppings for industrial floors. High-strength concretes with low *w/c* ratios are less dependent on aggregate type, and the use of a low *w/c* ratio can provide a dense, strong concrete that

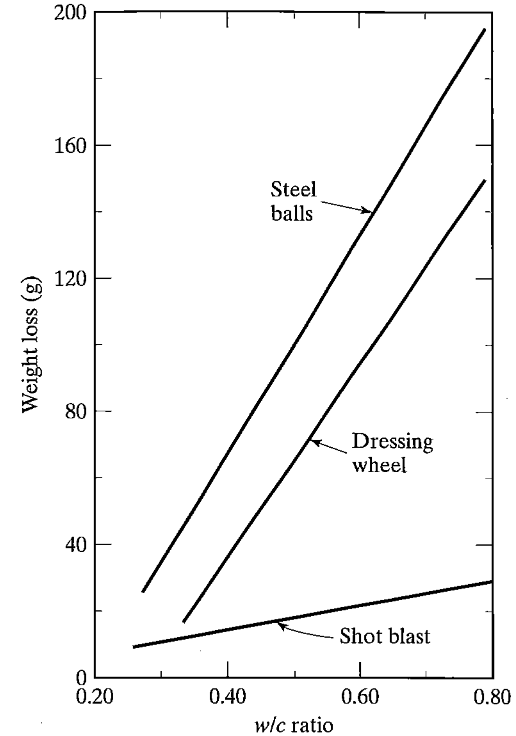


FIGURE 17.10

Influence of *w/c* ratio on the abrasion resistance of concrete. [From F. L. Smith in *Papers on Cement and Concrete*, ASTM STP 205, pp. 91–105 (1958). Copyright ASTM INTERNATIONAL. Reprinted with permission.

is resistant to abrasion (Figure 17.10), provided that correct finishing techniques are used. To provide a dense uniform finish, the surface should be steel troweled; more than one pass over the surface can be used. Special chemicals can be used to provide additional surface hardening. Such treatments involve impregnating the surface layers with a liquid, which subsequently reacts to deposit solid material in the pores of the paste. Many formulations are based on sodium silicate, which reacts with calcium hydroxide to form C–S̄–H or on soluble fluorosilicates, which form insoluble calcium fluorosilicate.

It is difficult to assess the abrasion resistance of concrete, since the damaging action depends on the exact cause of wear. Wear on pavements and floors includes scraping damage and impact loading as well as frictional attrition. Several tests have been prepared using different kinds of wear, but no one test is satisfactory for evaluating all conditions of abrasion. Indeed, the most difficult part of an abrasion test may be interpreting the results. ASTM C 779 gives three optional methods for testing the abrasion resistance of horizontal concrete surfaces. The *steel-ball abrasion test* applies a load to a rotating head containing steel balls, while the abraded material is washed away by water to expose fresh surfaces. The *dressing wheel test* applies a load through rotating steel dressing wheels. The *revolving disk test* uses revolving flat steel disks in conjunction with a silicon carbide abrasive grit. In each of the tests, the depth of wear is measured after a set time of test; weight loss is also a common measure of wear. These tests are meant to simulate wear from wheeled and heavy foot traffic and apply various

amounts of frictional abrasion, together with impact and scuffing such as might occur in practice. The *sandblast test* (ASTM C 418) is probably a better measure of resistance of concrete to erosion, although it can be used as an abrasion test also. This test tends to abrade the less resistant components of the concrete more severely. In addition, there are tests for the abrasion resistance of aggregates, such as the *Los Angeles Machine test*, which was discussed in Chapter 7. The *rotating cutter* method in ASTM C 944 is used for quality control of pavements. It is similar to the dressing wheel test, but is designed for specimens, such as cores, that have test areas that are too small to run the tests in ASTM C 799 or C 418.

Special tests for aggregates may be required for particular applications; for example, skid resistance of pavements is an important consideration. The gradual polishing of aggregates will eventually cause a smooth skid-prone surface to develop. Hard, friable aggregates, which fracture rather than polish, may be the most desirable to maintain a skid-resistant surface. Aggregates which are made up of minerals that have different rates of polishing will also give a more skid-resistant surface. There are various ASTM tests designed to measure the skid resistance of pavement surfaces, including concrete. ASTM E 274 and ASTM E 303 are designed for field testing. ASTM E 274 uses a specified, full-scale automotive tire, while ASTM E 303 uses a special rubber slider in contact with the surface. ASTM E 303 can be adapted for laboratory testing, or ASTM E 510 can be used for this purpose. Both ASTM E 303 and ASTM E 510, as well as ASTM E 451, can be used to study the polishing characteristics of the pavement.

Erosion Resistance

The erosion of concrete will depend very much on the quantity and properties of suspended solids. Erosion by waters free of sediment is not likely to be great unless chemical attack can occur. Again, resistance to erosion is primarily determined by the compressive strength (or *w/c* ratio) of the concrete. The use of large aggregate particles seems to be advantageous. As in the case of abrasion, it is only the concrete at the surface that is subjected to erosion. Thus, particular attention should be paid to ensuring that a high-quality surface is attained during placement. While the sandblast test (see previous discussion) can be used to measure resistance to erosion, ASTM C 1138 is designed to simulate the wear caused by waterborne particles. The test apparatus consists of a rotating paddle that moves water within a steel cylinder mounted on the concrete test surface. Steel grinding balls of various sizes are driven on the surface of the concrete by the water. Abrasion resistance is judged based on the volume of material removed and the depth of wear.

Cavitation Damage

Damage by cavitation is a far more serious problem than erosion and even good-quality concrete may suffer severe attack. Cavitation occurs when a high-velocity flow of water (or any other fluid) suffers an abrupt change in direction or velocity. This causes a zone of low pressure to occur at the surface immediately downstream from the change, and this drop in pressure may be sufficient to allow pockets (or cavities) of vapor to form. These pockets of vapor collapse catastrophically on leaving the low-pressure zone, and their collapse causes a localized high-energy impact on the concrete surface. The impact from the implosion of vapor cavities may generate localized pressures as high as 700 MPa (100,000 lb/in.^{2}), which is sufficient to affect even the strongest metals. Clearly, concrete can be severely affected, and perhaps the best defense against cavitation is to try to eliminate its possibility in design. The elimination of irregularities in flow or the reduction of flows to less than 12 m/s (40 ft/s) will do much to help. Deliberate air entrainment in water is said to cushion the effect of cavitation and hence reduce damage.

Cavitation cannot always be avoided, however, and the use of very good materials should be considered in minimizing damage. Experience has shown that the use of a high-strength, low *w/c* ratio concrete with a small aggregate size (not greater than 20 mm or $^3/_4$ in.), and a good paste–aggregate bond will give the best resistance. The use of polymer-impregnated concrete (for high strength and good bond) or fiber-reinforced concrete (for good impact resistance) has been advocated for protection against cavitation damage. These newer materials (see Chapters 21 and 22) are now being tested in the field.

17.3 OTHER PROPERTIES

Radiation Shielding

Concrete has useful properties as a radiation-containment material. Its most visible application has been in prestressed concrete pressure vessels for nuclear reactors. Normal concrete can be used for shielding, but may require excessive thicknesses. The choice of suitable materials will improve the attenuation of radiation and reduce the required thicknesses. The principal types of radiation that need to be considered in the design of radiation shielding are neutrons and gamma rays.

Gamma radiation is highly penetrating, electromagnetic radiation with wavelengths much shorter than X rays. Gamma rays are attenuated primarily through elastic collision with electrons (the Compton scattering effect). High-density materials are good attenuators, and the use of special high-density aggregates is desirable. Attenuation of fast neutrons also requires a high density, while the presence of hydrogen atoms will cause moderate and slow neutrons to be absorbed. For normal-density concrete, a hydrogen content of 0.45% by weight is required, which is reached when the concrete contains about 4% by weight of water. Fully dried concrete will not contain sufficient water, so that an aggregate containing some hydrogen may also be desirable (Table 17.4). When hydrogen absorbs slow neutrons, high-energy gamma rays are produced, which must also be attenuated. Boron is a more effective absorber of neutrons and produces gamma rays with a much lower energy that are more readily shielded. Thus, boron-containing aggregates are useful for neutron shielding. The addition of water-soluble boron compounds is not advised, however, since they may retard the hydration of cement. Siliceous aggregates have a high proportion of oxygen, which also helps to moderate neutrons. Aggregates for radiation shielding are covered by ASTM C 637 and ASTM C 638. Heavy aggregates are more expensive and require special placing and mix design considerations. They should be used only when space limits the thickness of the concrete. Heavy aggregates tend to be harsh, with a tendency to segregate. Therefore, more fine aggregate than usual is needed; the use of fine sands and high cement contents will give workable, cohesive concretes. (See Chapter 20 for additional discussion of heavyweight concrete.)

TABLE 17.4 Aggregates Recommended for Radiation Shielding

Aggregate Type	*Specific Gravity*	*Shielding Capability*
Natural		
Bauxite	~2.0	Fast neutrons (H)[a]
Serpentine	~2.5	Fast neutrons (H)
Goethite	~3.5	Fast neutrons (H)
Limonite	~3.5	Fast neutrons (H)
Borocalcite	~2.5	Neutrons (B)[b]
Colemanite	~2.5	Neutrons (B)
Barite	~4.2	Gamma rays
Magnetite	~4.5	Gamma rays
Illmenite	~4.5	Gamma rays
Hematite	~4.5	Gamma rays
Synthetic		
Heavy slags	~5.0	Gamma rays
Ferrophosphorus	~6.0	Gamma rays
Ferrosilicon	~6.7	Gamma rays
Steel punching or shot	~7.5	Gamma rays
Ferroboron	~5.0	Neutrons (B)
Boron carbide	~2.5	Neutrons (B)
Boron frit	~2.5	Neutrons (B)

[a](H), moderation by hydrogen.
[b](B), moderation by boron.

Radiation Damage It appears that concrete properties are unaffected by exposure to gamma rays or neutrons at levels generally encountered. This statement is equivocal because it is difficult to separate the effects of radiation from the effects of temperature. The energy of absorbed radiation is converted to heat and results in quite large temperature increases in concrete −100°C (180°F) or more. The temperature of concrete in reactor shields may attain several hundred degrees unless steps are taken to lower temperatures by removal of heat. The safe operating temperature depends on many factors: the thermal stability of aggregates, the presence of temperature cycling (which is more damaging than a steady temperature), the size of temperature gradients, and the minimum shielding requirements.

Acoustic Properties

Two acoustic properties of building materials are of interest: sound absorption and sound transmission. Sound absorption is primarily concerned with controlling noise levels within a room. The *sound absorption coefficient* depends strongly on porosity; thus, lightweight concrete gives better sound absorption than does normal-weight concrete. Yet texture is also important, and a concrete made with porous, lightweight aggregate, which has an irregular, interconnected porosity absorbs sound better than a foamed concrete of the same porosity, which has discrete air bubbles.

The main factor influencing *sound transmission* is the density per unit area of wall; the type of material used is less critical. Low-density concretes are both good transmitters and absorbers of sound; thus, if both high absorption and low transmission are required simultaneously, some solution must be found, or a compromise must be made. The presence of cavities increases transmission loss, so the use of cavity walls or the use of sandwich panels with a dense interior may provide an acceptable solution. The use of gypsum board and furring strips to provide a cavity adjacent to a concrete wall will reduce sound transmission.

Sound transmission depends on the frequency of sound. Measurements of sound-transmission loss are made according to ASTM E 90 for a series of frequencies. From these measurements, a single-figure rating, the sound-transmission class, is determined according to ASTM E 413. Where thin partitions are used, the bending stiffness of the wall may be important. Under the right conditions, the air-wave frequency imposed on the wall may equal its structural resonance frequency, and this condition will enhance sound transmission.

Electrical Properties

Electrical Conductivity Dry concrete is a good insulator with a resistivity of about 10^{13} ohm · m. However, moist concrete has a much lower resistivity, about 10^6 ohm · m, which is in the range for semiconductors. The increase in conductivity is due to the presence of water in the capillary pores, which contains dissolved salts and so acts as an electrolyte. Therefore, the resistivity of concrete will be controlled by the concentration of the electrolytic solution. Factors that increase the concentration are the alkali content of the cement (alkali salts predominate in pore solutions), the presence of electrolytic admixtures such as calcium chloride, and the amount of capillary water, which is controlled by the degree of hydration and the *w/c* ratio. These effects are shown in Table 17.5.

This discussion applies to alternating current. The resistivity to direct current may be different because of a greater polarizing effect. Concrete offers a high resistance to the passage of current to or from reinforcing steel, believed to be due to polarizing effects.

Other Properties The capacitance of concrete decreases with age and increasing frequency. The dielectric strength of concrete is apparently independent of moisture content, but for a composite material such as concrete, it should depend on the microstructure of

TABLE 17.5 Effect of Alkali Content on Resistivity of Concrete[a]

Na_2O Content	*w/c Ratio*	*Resistivity (at 1000 Hz, 4 V) (ohm · m)* 7 Days	28 Days	90 Days
0.2%	0.4	103,000	117,000	157,000
	0.5	79,000	88,000	109,000
	0.6	53,000	70,000	76,000
1.0%	0.4	123,000	136,000	166,000
	0.5	82,000	95,000	120,000
	0.6	72,000	73,000	79,000

[a]Adapted from G. E. Monfore, *Journal of the Portland Cement Association*, R & D Laboratories, Vol. 10, No. 2, pp. 35–48 (1968).

the material. The dielectric properties of cement pastes have been studied in the radiofrequency range, and microwave absorption has been used to study the behavior of water in cement paste.

The impedance of cement paste and concrete is the result of concomitant capacitance of the solid phase and parallel conductivity through the pore solutions. The effects vary with frequency under AC and give rise to an impedance spectrum. Analysis of the spectrum gives useful information about the transport properties of the material and may become the basis of future tests for predicting concrete performance.

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PROBLEMS

17.1. What is the significance of the fact that concrete aggregates and cement paste have different coefficients of thermal expansion?

17.2. Why are lightweight concretes good thermal insulators?

17.3. How would you determine whether a structure has been structurally damaged after exposure to fire?

17.4. Why is concrete more fire resistant than structural steel?

17.5. What factors affect the wear resistance of concrete?

CHAPTER 18

Durability

Concrete is inherently a durable material. If properly designed for the environment to which it will be exposed, and if carefully produced with good quality control, concrete is capable of maintenance-free performance for decades without the need for protective coatings, except in highly corrosive environments. Concrete, however, is potentially vulnerable to a variety of different exposures (Table 18.1) unless certain precautions are taken. Deterioration of concrete can be caused by the adverse performance of any one of the three major components (aggregate,[1] paste, or reinforcement) and can be due to either chemical or physical causes. Although one particular environmental factor may initiate distress, other factors may then contribute and aggravate the situation.

A major difficulty in studying durability is predicting concrete behavior several decades in the future on the basis of short-term tests. Much of our understanding of durability has come through a direct study of actual field problems. A major challenge facing the modern concrete technologist is the more accurate prediction of concrete durability under a variety of service conditions.

18.1 PERMEABILITY OF CONCRETE

The single parameter that has the largest influence on durability is the *w/c* (or *w/cm*) ratio. As the *w/c* ratio decreases, the porosity of the paste decreases and the concrete becomes more impermeable (Figure 18.1). The effect of variation in *w/c* ratio on permeability is dominated by "large" capillary porosity, rather than gel pores. The effect of *w/c* ratio on the volume of capillaries is shown in Figure 18.2. The figure represents the physical composition of a sealed, fully hydrated portland cement paste. The condition of a sealed paste (no additional water available after mixing) is representative of the interior of most high quality concrete structures. As shown in the figure, capillary volume increases markedly for *w/c* ratios greater than 0.42.

The permeability of concrete plays an important role in durability because it controls the rate of entry of moisture that may contain aggressive chemicals and the movement of water during heating or freezing. The *w/c* ratio has a dual role to play in

[1] See Chapter 7 for a discussion of the alkali–aggregate (alkali–silica) and alkali–carbonate reactions.

TABLE 18.1 Durability of Concrete

Chemical attack
Leaching and efflorescence (P)[a]
Sulfate attack (P)
Alkali–aggregate reaction (A) (Chapter 7)
Acids and alkalis (P)
Corrosion of metals (R)
Physical attack
Freezing and thawing (P, A)
Wetting and drying (P) (Chapter 16)
Temperature changes (P, A) (Chapter 17)
Wear and abrasion (P, A) (Chapters 7, 17)

[a]Letter(s) in parentheses indicates the concrete component most affected, in order of importance: A, aggregate; P, paste; R, reinforcement.

concrete durability since a lower *w/c* ratio also increases the strength of concrete and hence improves its resistance to cracking from the internal stresses that may be generated by adverse reactions.

The permeability of concrete can be measured by determining the rate of flow through a concrete slab. Since the porosity of concrete resides in the paste, the permeability of concrete should be controlled by the paste, but is significantly modified by internal cracking at the paste–aggregate interface. It should be noted that the flow of water through concrete is of interest in construction aside from considerations of durability. Impermeable concrete is required for water-retaining structures and construction below grade. The term *watertightness*, which is commonly used in the construction

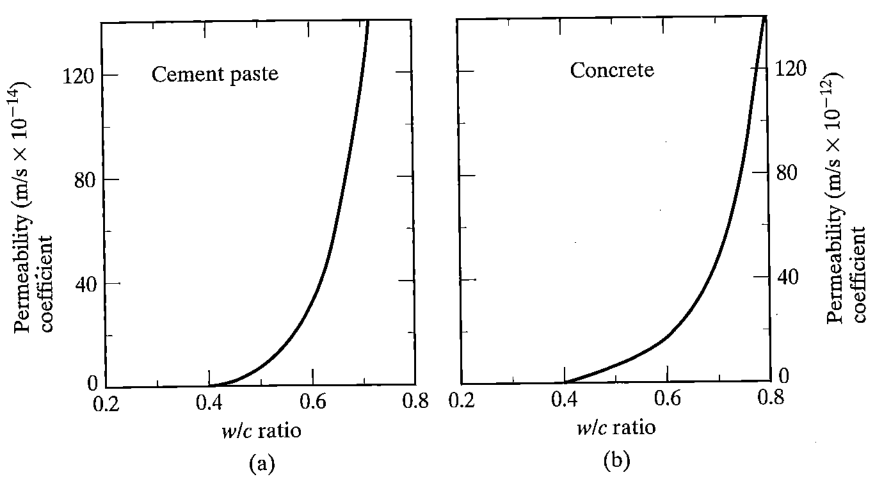


FIGURE 18.1

Influence of *w/c* ratio on the permeability of (a) cement paste [From T. C. Powers, L. E. Copeland, J. C. Hayes, and H. M. Mann, *Journal of the American Concrete Institute*, Vol. 51, No. 3, pp. 285–298 (1954)]; (b) concrete [adapted from *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, CO (1975)].

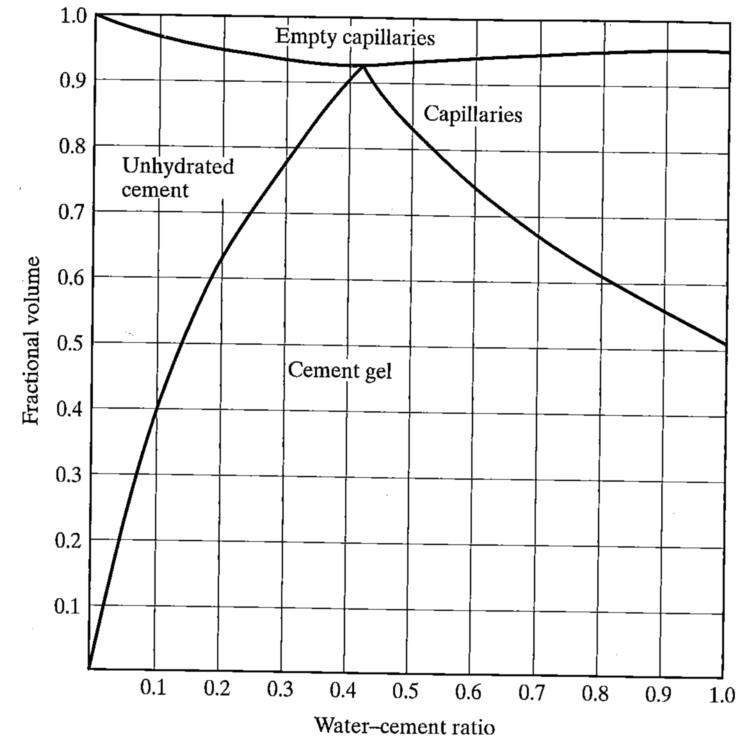


FIGURE 18.2

Composition of sealed and fully hydrated portland cement paste. [From T. C. Hansen, *Materials and Structures*, Vol. 19, No. 114, pp. 423–436 (1986).]

industry, is synonymous with impermeability. The flow of water through cement paste obeys *D'Arcy's law* for flow through a porous medium, namely,

$$v = K_p \frac{h}{x} \tag{18.1}$$

where v = the rate of flow of water, h = head of water (hydraulic pressure), x = thickness of the specimen, and K_p = permeability coefficient. K_p is not a constant for cement paste, being dependent on the *w/c* ratio and age of the paste, as can be seen in Table 18.2. This is because K_p is strongly dependent on the capillary porosity of the paste, which is controlled by both *w/c* ratio and degree of hydration. It can be seen from Table 18.2 and Figure 18.1 that K_p varies over several orders of magnitude. Concrete is typically 10 to 100 times more permeable to gas than to water.

Tests to specifically measure permeability usually fall into three categories. Two involve the movement of water through concrete, while the third involves the movement of electric charge. In the traditional method, a slab of concrete is subjected to a high water pressure, and the flow through the concrete is measured. A high head (pressure) is often used to obtain a measurable steady-state flow. One difficulty with this technique is that the value of K_p tends to decrease as the pressure increases. To obtain

TABLE 18.2 Effect of Age of Cement Paste on Its Permeability Coefficient $w/c = 0.51$)

Age (days)	K_p *(m/s)*	
Fresh paste	10^{-5}	Independent of *w/c*
1	10^{-8}	Capillary pores interconnected
3	10^{-9}	
4	10^{-10}	
7	10^{-11}	
14	10^{-12}	
28	10^{-13}	
100	10^{-16}	Capillary pores discontinuous
240 (maximum hydration)	10^{-18}	

a more realistic value of K_p, permeability tests should be run with a minimum pressure gradient (although this considerably increases the time needed to reach steady-state flow). The degradation in the gradient provides an indication of the flow through the concrete. A third option is to measure permeability using an indirect method—most often the so-called rapid chloride permeability test, or RCPT (ASTM C 1202—the full title is the "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration"). The test involves the application of a voltage between two sides of a concrete specimen with solutions of sodium hydroxide and sodium chloride on opposite sides. The total charge passed during a six-hour period provides a measure of the permeability. For concretes with *w/c* ratios between 0.4 and 0.75, results from the RCPT correlate well with both of the more conventional pressure methods, as shown in Figure 18.3, and total porosity. However, since the

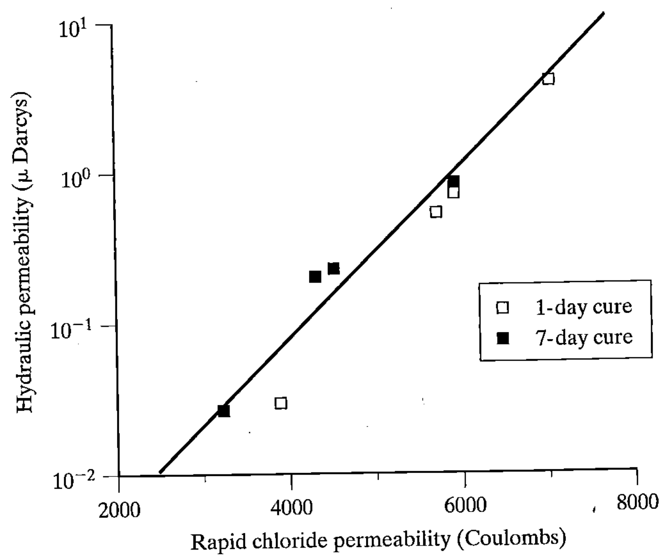


FIGURE 18.3

Relationship between results of hydraulic permeability tests and rapid chloride penetration tests (RCPT). [From D. Whiting in *Permeability of Concrete*, SP-108, American Concrete Institute, Detroit, MI, pp. 195–222 (1988).]

RCPT actually measures the electrical charge passing through a specimen, test results must be evaluated carefully. Low-permeability concretes exhibit a nearly constant current during the six-hour test, while high-permeability concretes can exhibit steadily increasing current, due to heating (caused by the initially higher current) which tends to increase conductivity. Thus, the initial current or the charge passed in the first half-hour of the test often provides more objective comparisons of concretes than do the results for the full six-hour test. Low-permeability concretes containing silica fume tend to exhibit considerably lower RCPT readings than similar concretes without silica fume. In these comparisons, however, the differences in RCPT readings indicate differences that may be several times greater than differences observed in tests involving actual chloride penetration.

All permeability measurement methods exhibit significant variability (coefficient of variation can be 30 to 50%) and cannot provide quantitative data for concretes with low values of permeability ($K_p \leq 10^{-15}$ m/s), such as high-strength concretes and concretes containing supplementary cementing materials.

In a mature cement paste, the permeability coefficient is very small, even though the total porosity is high, and is of the same order as that observed in low-porosity rocks. Thus, it can be concluded that water does not easily move through very small gel pores and that permeability is controlled by an interconnecting network of capillary pores. As hydration proceeds, the capillary network becomes increasingly tortuous as interconnected pores are blocked by the formation of C–S–H. This is accompanied by a continuous decrease in K_p, and the time at which complete discontinuity of the capillary of pores occurs is a function of the *w/c* ratio (Table 18.3). In concretes with a *w/c* ratio greater than 0.70, complete discontinuity of capillary pores can never be achieved, even with continuous moist curing, and concretes will have relatively high permeabilities. Even after the capillary pores have been completely isolated by regions of C–S–H and its attendant gel porosity, K_p continues to decrease by several more orders of magnitude. This is due not only to an increase in thickness of C–S–H between capillaries, but also to the fact that calcium hydroxide continues to grow within the residual capillary pores, thus forming impermeable regions. A limiting value of K_p should occur when all capillary porosity has been eliminated, and this is believed to be less than 10^{-22} m/s.

TABLE 18.3 Curing Time Required to Produce a Discontinuous System of Capillaries (Assuming Continuous Moist Curing)

w/c Ratio	*Curing Time (days)*
0.40	3
0.45	7
0.50	28
0.60	180 (6 months)
0.70	365 (1 year)
> 0.70	Not possible

The size of the interconnected capillary system will influence the magnitude of K_p. There is a good experimental correlation between K_p and the critical effective pore access diameter (d_c) determined by mercury intrusion porosimetry (see Section 4.3). The value of d_c corresponds to the maximum rate of intrusion (dV/dlog$d$) of the intrusion curve. Computer simulation predicts interconnectivity of the interfacial porosity, and this has been observed experimentally by mercury intrusion. The larger value of d_c associated with the interfacial porosity increases the value K_p by a factor of 10–100.

The foregoing discussion refers to well-compacted pastes that are continuously moist-cured. If pastes are allowed to dry and then are rewetted, the permeability coefficient is higher. This may be due to changes in pore-size distributions that occur on shrinkage and allow capillary pores to become partially interconnected again. The effect is more marked in concrete, since cracking at the paste–aggregate interface will create further opportunities for water flow. The importance of length of curing is emphasized by the observation that, at an age of three months, concrete that has been wet cured for seven days has a permeability that is one-fifth the value for the same mix wet cured for one day.

Even in continuously saturated concretes, permeability is increased by imperfect consolidation or excessive segregation of materials, which can create bleeding channels within the paste. An increase in entrained air of 1% has been shown to have the same effect on permeability as an increase in *w/c* ratio of 0.03. However, the extra workability provided by entrained air (as well as the need to maintain strength) allows for a decrease in water content. The subsequent decrease in *w/c* ratio, along with the reduction in segregation and bleeding obtained with entrained air, results in an overall reduction in permeability.

It has been widely observed that the addition of supplementary cementitious materials, especially silica fume, results in a significant decrease in permeability—a greater reduction in fact than can be attained from the effect on the *w/cm* ratio alone. This is, in large part, due to the reduction both in total porosity and in the size of the pores. Scanning electron microscope studies show that silica fume significantly decreases the number of pores with diameters between 1 and 10 μm, while mercury intrusion porosimetry shows that silica fume reduces the number of pores with diameters between 0.010 and 1.0 μm.

The effect of cracking or separation at the interface between paste and aggregate is demonstrated by the effect of maximum coarse aggregate size on permeability. The data used to develop Figure 18.1b show that, at a *w/c* ratio of 0.5, an increase in the maximum aggregate size from 37.5 to 115 mm ($1^1/_2$ to $4^1/_2$ in.) increases the permeability of concrete from 2.4 to 6.8 $\times$ 10^{-12} m/s (0.8 to 2.2 $\times$ 10^{-11} ft/s). Mass concrete used in dams is typically placed at a higher *w/c* ratio and may have permeabilities between 8 and 35 $\times$ 10^{-12} m/s (2.6 to 11.5 $\times$ 10^{-11} ft/s). The U.S. Bureau of Reclamation has adopted K_p = 15 $\times$ 10^{-12} m/s (4.8 $\times$ 10^{-11} ft/s) as a maximum permeability for some of its work.

18.2 CHEMICAL TRANSPORT

Most concrete contains capillary water; therefore, chemical transport will invariably be affected by the interaction between pore solution and the chemical. Acids will dissolve the components of cement paste and some aggregates and, thus, increase the permeability and diffusion coefficients of concrete. Some organics will form calcium soaps, which will have the effect of decreasing permeability, but most hydrocarbons do not react with concrete. Some inorganic salts, such as sodium and calcium chloride, may have a physical effect on concrete by increasing the severity of freeze–thaw cycles or may interact with reinforcing steel, causing corrosion. Others, containing sulfates, may cause severe damage due to chemical attack.

The effect of moisture content on the penetration of an organic liquid is shown in Figure 18.4. In this case, the penetration of butanol into dry concrete (upper curve) is 2 to $2\frac{1}{2}$ times greater than the penetration into concretes containing 2% moisture by mass (lower curves).

Moisture content largely determines the nature and speed of penetration of any chemical into concrete. Dissolved ions will move with water as it flows under pressure head according to D'Arcy's Law [Eq. (18.1)]. For dry (or semi-dry) concrete exposed to water, capillary suction pressure σ has the same effect on flow as a pressure head of 2.4 MPa (400 psi) in saturated concrete. D'Arcy's Law applies to the penetration of other fluids as well; K_p may change, since the viscosity of the fluid is embedded in this constant.

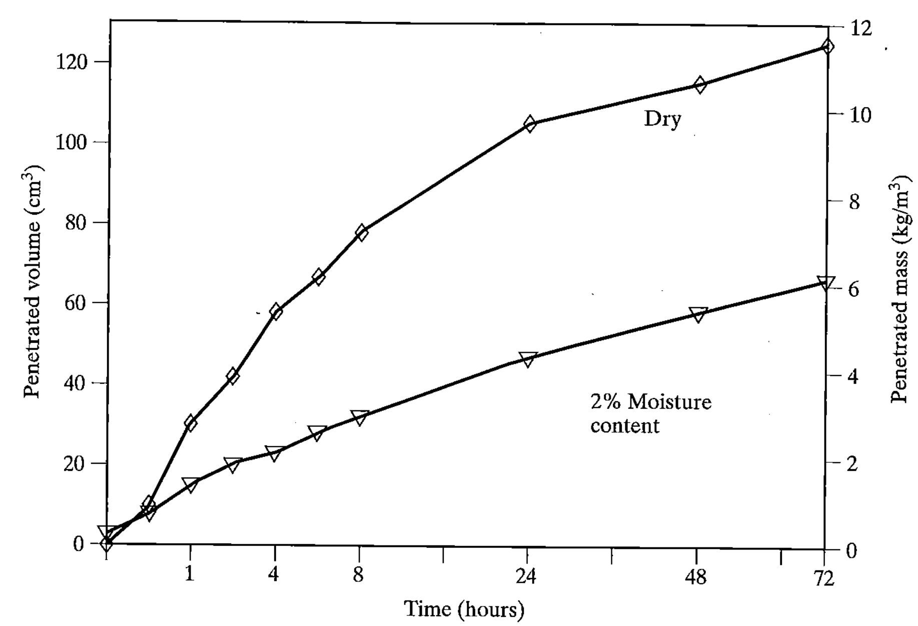


FIGURE 18.4

Absorption of butanol in concretes with different moisture contents. [Adapted from H. W. Reinhardt in *Materials Science of Concrete III*, ed. J. Skalny, pp. 209–242 (1992). Reprinted with permission from the American Ceramic Society, Westerville, OH]

In near-saturated concrete where no pressure head exists, diffusion under a concentration gradient provides the principal method of transport. For most applications, this diffusion can be described by *Fick's Second Law*:

$$\frac{\partial C}{\partial t} = K_d \frac{\partial^2 C}{\partial x^2} \tag{18.2}$$

in which C = concentration, t = time, K_d = diffusion coefficient (or diffusivity), and x = depth. The solution to this equation for a semi-infinite slab is

$$C(x, t) = C_o\left[1 - \mathrm{erf}\left(\frac{x}{2\sqrt{K_d t}}\right)\right] \tag{18.3}$$

in which $C(x, t)$ = chloride concentration at depth x and time t, C_o = chloride concentration at the surface, and erf = the error function.

The penetration of chlorides, for example, follows Fick's Law very closely, as shown in Figure 18.5. The solution to Eq. (18.3) shows that the depth of penetration of a chemical by diffusion is proportional to $\sqrt{t}$. This relationship works just as well for the depth of carbonation (penetration of carbon dioxide) as for the penetration of chlorides and many other chemicals.

Research has shown that diffusivity can be related to electrical conductivity (i.e., the flow of current under an electrical potential = the reciprocal of resistivity) by the Nernst–Einstein equation,

$$\frac{K_d}{K_o} = \frac{\kappa}{\kappa_o} \tag{18.4}$$

where K_o and κ_o are the diffusivity and conductivity, respectively, of the pore solution, while K_d and κ are the values for concrete. It can be shown that

$$\frac{\kappa}{\kappa_o} = \beta\phi \tag{18.5}$$

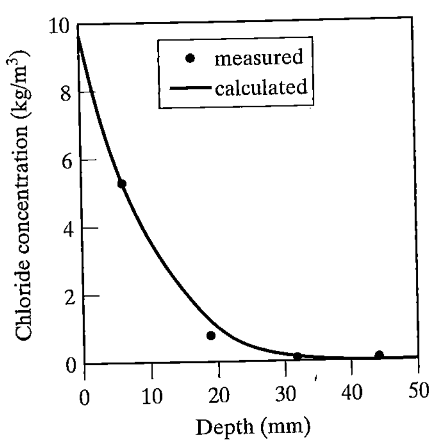


FIGURE 18.5

Chloride concentration profile following Fick's Second Law for parking garage in northern midwest USA. [From N. S. Berke and M. C. Hicks in *Techniques to Assess the Corrosion Activity of Steel Reinforced Concrete Structures*, ASTM STP 1276, pp. 41–57 (1996). Copyright ASTM INTERNATIONAL. Reprinted with permission.

where ϕ is the capillary porosity of the concrete and β is the tortuosity of the capillary pore network. Electrical conductivity is much easier to measure directly than diffusivity. K_o and κ_o represent the chemical contribution to transport, while $\beta\phi$ is the physical resistance of the pore network.

18.3 CHEMICAL ATTACK

Leaching and Efflorescence

Efflorescence occurs quite frequently on the surface of concrete when water can percolate through the material, either continuously or intermittently, or when an exposed face is alternately wetted and dried. Efflorescence consists of deposited salts that are leached out of the concrete and are crystallized on subsequent evaporation of the water or interaction with carbon dioxide in the atmosphere. Typical salts are sulfates and carbonates of sodium, potassium, or calcium, the major constituent being calcium carbonate.

Efflorescence, in itself, is an aesthetic rather than a durability problem, but it does indicate that substantial leaching is occurring within the concrete. Extensive leaching causes an increase in porosity, lowering the strength of the concrete and increasing the permeability, thereby increasing its vulnerability to aggressive chemicals. Calcium hydroxide is the hydration product that is most readily leached from concrete; C–S–H is essentially insoluble and is only decomposed under exposure to severe leaching conditions for long periods of time. Thus, pastes that have a high content of calcium hydroxide are likely to be more prone to leaching and efflorescence and to have a greater potential for deterioration in unfavorable conditions. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating water. Soft waters, such as rainwater, are the most aggressive, while hard waters containing large amounts of calcium ions are less dangerous. The temperature of the water is also a consideration, since calcium hydroxide is more soluble in cold water than it is in warm water. Leaching is, of course, most prevalent when the water can seep through the concrete, particularly under pressure. Concrete is not significantly leached by water flowing over its surface unless accompanied by physical abrasion from suspended solid matter.

Sulfate Attack

Perhaps the most widespread and common form of chemical attack is the action of sulfates on concrete. Sulfates are often present in groundwaters, particularly when high proportions of clay are present in the soil, and seawater has sulfates as a major constituent. Groundwaters may have local concentrations of sulfate in the vicinity of industrial wastes such as mine tailings, slag heaps, and rubble fills. Sulfates present in rainwater from air pollution, or produced by biological growths, may cause slow deterioration even in concrete above ground (Figure 18.6).

Mechanisms of Sulfate Attack Sulfate attack is actually a rather complex process that may involve all of the hydration products produced by portland cement. The damage caused by sulfate attack may involve cracking and expansion of concrete as a whole, as well as softening and disintegration of cement paste.

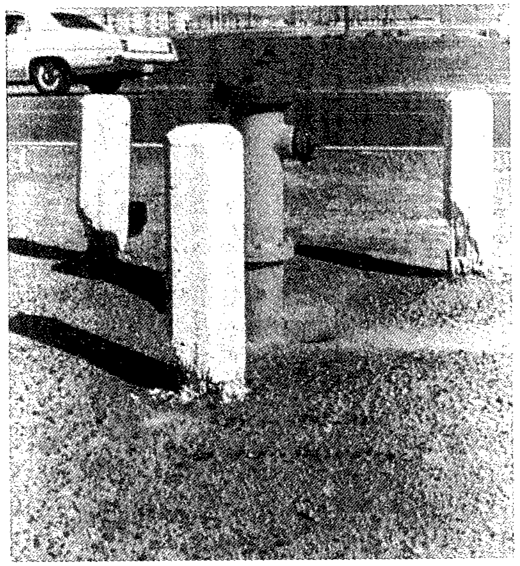


FIGURE 18.6

Concrete posts damaged near the soil line by sulfate attack. (Photograph courtesy of Portland Cement Association.)

Of the various forms of attack, laboratory and field experience has established a clear correlation between the C_3A content of a portland cement and its susceptibility to sulfate attack. As discussed in Chapter 4, cements with a high C_3A content will be subject to *sulfoaluminate corrosion,* in which ettringite is formed from monosulfoaluminate:

$$C_4A\bar{S}H_{12} + 2C\bar{S}H_2 + 16H \rightarrow C_6A\bar{S}_3H_{32} \qquad (4.5)$$

This is accompanied by a 55% increase in solid volume, causing a volume expansion within the paste that generates accompanying internal stresses and ultimately leads to cracking. Volume expansion may also be increased due to water absorption when the ettringite is in a microcrystalline form.

Sulfoaluminate attack is initiated by the reaction between sulfate ions and calcium hydroxide:

$$CH + SO_4^{2-}(aq) \rightleftarrows C\bar{S}H_2 + 2OH^-(aq) \qquad (18.6)$$

This reaction can be described as *gypsum corrosion,* since it is accompanied by an expansion in solid volume of about 120%. At high exposures to sulfates, gypsum corrosion appears to play a secondary role compared to that of the reaction in Eq. (4.5) for periods of 10 or more years. With long-term exposure, however, gypsum corrosion can be a primary mechanism of damage even for portland cements with low aluminate concentrations. [Although the percentage increase in volume obtained with the reaction in Eq. (18.6) is greater than that for the reaction in Eq. (4.5), the volume of monosulfoaluminate is far greater than that of the other constituents shown in the equations, and, thus, the total volume expansion resulting from Eq. (4.5) is greater than that obtained from Eq. (18.6).] At low sulfate concentrations, the importance of Eq. (18.6) is that it encourages the penetration of sulfate ions into the concrete and concentrates

them in a form in which they can react directly with monosulfoaluminate. Sulfate attack is not always accompanied by large expansion, and field observations indicate that, in most cases, the cement paste has undergone significant softening and disintegration involving decalcification of C–S–H, leaving the cement paste in a puttylike state. The attack on C–S–H depends largely on the particular sulfate involved.

Effect of Different Sulfates With relatively small concentrations of sulfate ions, the nature of the accompanying cations does not affect the process of sulfate attack. With increasing concentrations of sulfate ions, this is no longer true, although in all cases the rate of sulfate attack has a less than linear relationship with sulfate concentration. At intermediate concentrations (1,000 to 2,000 ppm), calcium sulfate may appear more aggressive because of a faster rate of gypsum corrosion, but its effect is limited by the relatively low solubility of gypsum. At the increasingly higher sulfate concentrations that can be achieved with more soluble sulfates, gypsum corrosion has an increasing contribution. Magnesium sulfate can be even more aggressive because of the possibility of additional corrosion reactions due to the presence of magnesium ions, which can decompose both C–S–H and the calcium sulfoaluminates:

$$C_3S_2H_3 + 3M\bar{S}\,(aq) \rightarrow 3C\bar{S}H_2 + 3MH + 2SH_x \qquad (18.7)$$

$$C_4A\bar{S}H_{12} + 3M\bar{S}\,(aq) \rightarrow 4C\bar{S}H_2 + 3MH + AH_3 \qquad (18.8)$$

The silica gel formed, as in Eq. (18.7), may react slowly with MH (brucite) to form a crystalline magnesium silicate, which has no cementing properties. At very high concentrations of $MgSO_4$, sulfoaluminate corrosion is totally replaced by magnesium corrosion.

Equations (18.7) and (18.8) proceed because of the insolubility of magnesium hydroxide. Precipitation of MH also increases the rate of gypsum corrosion and decreases the pH of the pore solution:

$$CH + M\bar{S}\,(aq) \rightarrow C\bar{S}H_2 + MH \qquad (18.9)$$

The increase in solid volume is greater in Eq. (18.9) than in Eq. (18.6), but the formation of MH is such that it often forms in the existing pores without causing disruptive expansions. When this happens in low *w/c* ratio concretes, it tends to seal the concrete and further hinder penetration of sulfates.

Overall, sulfate attack may be considered as a sequence of three processes:

1. The first process is the diffusion of sulfate ions into the pores of concrete, which is controlled by the permeability coefficient (K_p) and the diffusion coefficient of the sulfate ions (K_d).
2. In its initial stages, gypsum corrosion may actually be beneficial, since gypsum is more soluble than calcium hydroxide and the dissolution–crystallization reaction will allow gypsum first to crystallize without expansion. This may explain why sulfate attack on cement pastes with little or no C_3A takes decades, rather than years, to occur. The extent of gypsum formation will depend on whether the hydroxide ions are removed, either by water moving through the concrete (in which case, gypsum will also be leached out) or by precipitating as another insoluble compound.

3. As sulfoaluminate corrosion causes internal cracking, the effective K_p will be increased, thereby accelerating further sulfate attack. Similar behavior is also expected for cements with low C_3A contents subjected to sulfate attack. However, the time over which severe expansion occurs due to attack on CH and C–S–H will be extended compared to cements high in C_3A.

When sulfate concentrations in the pore solution are low, ettringite can dissolve and recrystallize as large crystals in large pores, voids, or cracks. The recrystallization can occur over time in regular concrete and does not necessarily indicate sulfate attack is occurring.

Action of Seawater The protective nature of magnesium hydroxide is one reason why seawater is less corrosive than might be expected from a consideration of typical salt concentrations (see Table 6.2, Section 6.2). Another reason is that gypsum and ettringite are more soluble in solutions containing the chloride ion, another major constituent of seawater, and this reduces deleterious expansions. Thus, concretes remain intact, although strength may decrease slowly, leaching. Deterioration by seawater is greatest in the intertidal zone, where additional destructive mechanisms can occur. Frequent wetting and drying will aggravate the effect of sulfate attack, while the crystallization of sea salts in the concrete on evaporation may also contribute to expansive forces. Further damage may be caused by frost action and is aggravated by wave impact and abrasion by floating debris. Furthermore, the chloride ion will cause severe corrosion of the reinforcing steel if this is not controlled by the use of low-permeability concrete.

Thaumasite Formation One form of sulfate attack that has not been discussed involves the breakdown of C–S–H in the presence of sulfate and carbonate ions, followed by the formation of thaumasite, $C_3S\bar{C}\bar{S}H_{15}$. Thaumasite has a crystal structure that is similar to that of ettringite and often forms in cold, wet environments, typically when temperatures are below 15°C (60°F), and drops off as the temperature rises to 20°C (68°F). The carbonate ions may be the result of carbonation of the concrete or, in a number of well-documented cases, the use of either limestone coarse aggregate or limestone filler in portland cement. The thaumasite form of sulfate attack culminates with conversion of the concrete to what can be best described as mush. Thaumasite formation has been observed to increase with an increase in C_3A. Portland cements with 70 and 90% replacement by granulated blast furnace slag have performed well under conditions in which portland cement alone or portland cement with 25 and 40% replacement levels of fly ash have performed poorly.

Control of Sulfate Attack Protection against sulfate attack requires both a high-quality concrete and a cement that is low in C_3A. The specific criteria depend on the level of exposure, as shown in Table 10.7(Section 10.3), which summarizes the requirements of the ACI Building Code. If the sulfate content is less than 0.10% in soils or 150 ppm in water, no special precautions are needed. For "moderate" sulfate exposure, which includes exposure to seawater, soil with water-soluble sulfates between 0.10 and 0.20%, and water with a sulfate concentration between 150 and 1,500 ppm, a cement with a

maximum of 8% C_3A must be used. For this case, the *w/cm* ratio is limited to 0.5 and the compressive strength must be at least 28 MPa (4000 psi). For higher sulfate exposure, Type V sulfate-resistant cement (C_3A content below 5.0%) or Type V cement plus a pozzolan must be used. The *w/cm* ratio and compressive strength requirements are also changed accordingly. As indicated in Table 10.7, pozzolans and blast furnace slag or blended cements containing these materials will improve sulfate resistance.

When selecting cementitious materials, the goals should be (1) full conversion of C_3A to ettringite early in the life of the concrete so that the reaction in Eq. (4.5) (Section 4.1) cannot occur and (2) conversion of CH to C–S–H so that the reactions in Eqs. (18.6) and (18.9) cannot occur. The use of either a cement low in C_3A (Type II or Type V) or Type K cement (an expansive cement—see Chapter 3) will satisfy the first goal, because ettringite will be maintained as the stable sulfoaluminate hydrate. The use of pozzolans or blast furnace slag will satisfy the second goal.

While using a sulfate-resistant cement is critical to controlling sulfate attack, the use of such a cement will not guarantee sulfate resistance if the concrete is highly permeable. Easy access of water to the cement paste still leaves CH and C–S–H open to attack, as indicated in Eqs. (18.6)–(18.9). Obtaining low permeability requires a low *w/cm* ratio (as emphasized in Figure 18.7), adequate compaction, thorough curing, and design for minimum cracking. Indeed, studies such as shown in Figure 18.7 demonstrate that a low *w/cm* ratio is *more* important than a low C_3A content in controlling sulfate attack.

Another method of protecting concrete against sulfate attack is applicable to precast members. In one study, steam curing at 175°C (350°F) for six hours significantly improved sulfate resistance. Concrete cured in this manner exhibited little effect of severe sulfate exposure for a period of 17 years. Curing at lower temperatures, however, provided much less protection. Steam curing, with the use of a silica addition, removes calcium hydroxide from hydrated pastes, and calcium sulfoaluminate hydrates are no longer stable phases. The alumina is incorporated into C–S–H and may form some

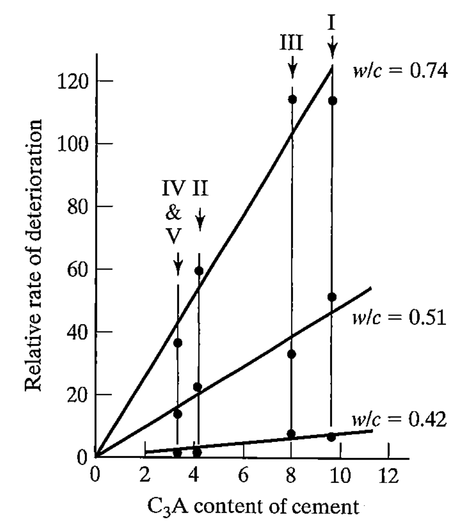


FIGURE 18.7
Effect of *w/cm* ratio and average C_3A content for different types of cement on the rate of deterioration under exposure to sulfate soils. [Adapted by G. Hill from G. J. Verbeck in *Performance of Concrete*, ed. E. G. Svenson, Univ. of Toronto Press, pp. 113–124 (1969).]

C_3AH_6, which is much more resistant to sulfate ions. More stable crystalline calcium silicate hydrates are also formed at higher temperatures.

Delayed Ettringite Formation Curing at the elevated temperatures used in many precast plants [70–88°C (158–190°F)] can have negative implications involving sulfates. High curing temperatures destroy ettringite—the sulfate, as well as the aluminate, appears to be absorbed by the C–S–H. After cooling, the sulfate is again available to form ettringite, with the associated increase in volume. The expansion results in the formation of macrocrystalline ettringite adjacent to aggregate particles. This form of sulfate attack, known as *delayed ettringite formation* or DEF, is distinctly different from that described earlier. DEF is usually limited to cases in which the cement has an SO_3 to Al_2O_3 ratio above 0.5 and the concrete is exposed to significant moisture. The specific mechanism of DEF is still not fully understood, but it does appear that incipient damage from other causes (e.g., ASR or thermal gradients) is a necessary component for DEF to cause deleterious expansions. The likelihood of DEF can be reduced by controlling the cement composition, using pozzolans and entrained air, and limiting both the maximum curing temperature to 70°C (158°F) and, to the extent possible, exposure to moisture.

Tests for Sulfate Attack Two ASTM tests provide a measure of the sulfate resistance of concrete. Neither, however, provides rapid results, and many different approaches have been used. ASTM C 452, which is proposed for research purposes only (Section 3.5), uses an abnormally high gypsum content in cement to test for sulfate resistance. It is a test for the cement only, not for concrete. This procedure has several drawbacks, the major objections being that the test is too slow for use as a routine specification test, that the behavior of concrete does not necessarily parallel the behavior of mortar bars, and that realistic exposure conditions are not reproduced. The test is only for use with portland cement and does not assess the behavior of cements blended with pozzolans or slags. ASTM C 1012 is designed to provide a means for evaluating the "expected" sulfate resistance of concretes and mortars made using portland cement, blends of portland cements with pozzolans or slags, and blended hydraulic cements. In this case, a mortar bar is exposed to a solution of sodium sulfate or a mixture of sodium sulfate and magnesium sulfate. Expansion is measured over time, and the value at six months is used as an acceptance criterion for sulfate resistance for blended cements (ASTM C 595). Clearly, significant work is needed to produce a rapid test for sulfate resistance.

Attack by Acids and Bases

Hydrated cement paste is an alkaline material; therefore, specific attack by other alkaline materials will not normally be encountered. High concentrations of alkaline materials that may come in contact with concrete in industrial processes cause deterioration by processes other than direct chemical reaction with hydroxide ions. The situation is entirely different for acidic solutions, which will readily attack basic materials such as concrete. Generally, naturally occurring acidic groundwaters are not common, being confined to marshy or peaty regions, where extensive decomposition of organic matter occurs. Acidic waters may also occur in, or adjacent to, landfilled areas and in places where mining operations and stockpiling of mine tailings have occurred. Highly acidic

conditions may exist in agricultural and industrial wastes, particularly from the food and animal-processing industries.

The hydrogen ion will accelerate the leaching of calcium hydroxide:

$$Ca(OH)_2 + 2H^+ \rightarrow Ca^{2+} + 2H_2O \tag{18.10}$$

If the ion is highly concentrated, C–S–H may also be attacked, forming silica gel:

$$3CaO \cdot 2SiO_2 \cdot 3H_2O + 6H^+ \xrightarrow{H_2O} 3Ca^{2+} + 2(SiO_2 \cdot nH_2O) + 6H_2O \tag{18.11}$$

The nature of the anion that accompanies the hydrogen ion may further aggravate the situation. Both sulfuric acid and carbonic acid are common constituents of acid groundwaters. The sulfate ion will obviously participate in sulfate attack, and thus sulfuric acid is particularly corrosive. Carbonic acid can also be very corrosive because of the formation of soluble calcium bicarbonate:

$$Ca(OH)_2 + H_2CO_3 \rightarrow Ca(HCO_3)_2 + H_2O \tag{18.12}$$

Any acid that can form soluble calcium salts in a like manner will be particularly aggressive, while if the acid forms an insoluble calcium salt, its precipitation during acid attack can protect the concrete against further deterioration. Some examples are given in Table 18.4.

Sugar, although not an acid, is another substance that dissolves more than just calcium hydroxide; both C–S–H and the calcium aluminate hydrates will be slowly attacked. For this reason, sugar in solution is very aggressive to concrete and should not be allowed to come in direct contact for more than brief periods.

It should be noted that corrosive chemicals can only attack concrete when water is present. Thus, concrete can be used to store dry chemicals.

Crystallization of Salts

We have been concerned hitherto with chemical attack of concrete, but salts can also cause damage to concrete through the development of crystal growth pressures that arise through physical causes. Corrosion of this type occurs when concrete is placed in

TABLE 18.4 Acid Attack of Concrete

Acid	*Formula*	*Likely Occurrence*
	Aggressive Acids That Form Soluble Calcium Salts	
Hydrochloric acid	HCl	Chemical industry
Nitric acid	HNO_3	Fertilizer manufacture
Acetic acid	CH_3CO_2H	Fermentation processes
Formic acid	$H \cdot CO_2H$	Food processing and dyeing
Lactic acid	$C_2H_4(OH) \cdot CO_2H$	Dairy industry
Tannic acid	$C_{76}H_{52}O_{46}$	Tanning industry, peat waters
	Acids That Form Insoluble Salts	
Phosphoric acid	H_3PO_4	Fertilizer manufacture
Tartaric acid	$[CH(OH) \cdot CO_2H]_2$	Winemaking

contact with water containing considerable quantities of dissolved solids. These salts will permeate into the concrete and will crystallize in the pores as the salts are concentrated by evaporation. Repeated or continuing evaporation will cause salt deposits to build up to the point where they will cause cracking. This problem is observed in regions between fluctuating water levels, for example in the intertidal region in marine exposures, but it can also occur when concrete is in contact with groundwaters rich in salts. In such cases, capillary effects can cause the salt-laden water to rise several hundreds of millimeters above the water level. If the water table is close to the ground surface, the capillary rise zone may extend above ground, where evaporation will take place most readily. Thus, damage generally occurs just above ground level. Damage of this type is found to be severe in certain areas of the Middle East, where high salt levels and high rates of evaporation make the problem acute. The problem also occurs in southern California where the floor slabs of some houses have been severely damaged. The soil contains a high concentration of salts, particularly sulfates, but the visible damage above ground is primarily the result of salt crystallization, rather than classical sulfate attack, which is found in the footings and stem walls. The cement in the slabs is generally low in C_3A, but the *w/c* ratio in the damaged slabs is well above 0.50. Capillary rise will also aggravate crystallization problems that occur with fluctuating water levels above ground.

One of the best ways to limit the problem of salt crystallization is to use a low *w/c* ratio–low permeability concrete, which limits moisture penetration. The problem can be corrected by sealing the concrete, either to prevent ingress of moisture or subsequent evaporation. A barrier can be built into the structure to prevent capillary effects from occurring. Alternatively, concrete below ground may be surrounded by an impervious clay fill to keep salts from coming in contact with the concrete, and in some areas, a sacrificial layer of mortar is placed on the exposed surface. In the latter case, crystallization damages the mortar, which can be replaced, rather than the underlying concrete.

Corrosion of Sewer Pipes

Concrete sewer pipes may be exposed to severe corrosion conditions. Not only may the pipes be corroded by substances in the groundwater (sulfates or acids), but they are also more prone to corrosion from the sewage itself. Furthermore, the thinness of the concrete pipe wall can make the effect of corrosion critical in a much shorter time. If an unbalanced water pressure exists, flow of water through the pipe wall may cause problems of leaching or salt deposition if wall thicknesses are less than 300 to 400 mm (12 to 16 in.). The *w/c* ratio of the concrete will obviously be an important parameter.

Domestic sewage or animal manures are usually harmless to concrete unless the reinforcing steel is inadequately protected. Acids or organic chemicals in industrial wastes can cause problems, as discussed earlier. However, the greatest problem in sewer pipes is corrosion caused by bacteria. The metabolic activity of many bacteria forms acids that may attack concrete, but the most harmful result of bacterial action in sewers is the formation of hydrogen sulfide in the sewage from the reduction of sulfate compounds by anaerobic bacteria. The hydrogen sulfide gas dissolves in water films in the upper part of the pipe, where there will be sufficient air to allow oxidation back to sulfate compounds by aerobic bacteria. In this way, high concentrations of sulfuric acid can be formed locally, which can corrode the upper part of the pipe very rapidly. Because cause of its ability to neutralize acids, limestone aggregate tends to provide concretes with a service life that is several times longer than that of concrete made with siliceous aggregate.

Protective treatment of sewer pipe surfaces will prolong the life of installations. Coatings of bitumens, tars, resins, and calcium aluminate cements have been used. Chemical treatments of the concrete surfaces are another possibility, and several are available that, when exposed to the concrete, precipitate insoluble salts to fill the pores or provide a resistant surface coating. Examples are water glass (sodium silicate), insoluble soaps, fluoride salts, and iron compounds. The surfaces can also be treated with carbon dioxide gas or silicon tetrafluoride vapor (the Okrat process). These gases form highly insoluble $CaCO_3$ or CaF_2, respectively, which seal the surfaces. For long-term protection, thick linings (polyvinyl chloride sheets) or coatings (epoxy resin mortars) are used. Often the most effective solutions, however, involve such steps as treating the sewage by chlorination to prevent formation of the sulfuric acid, adding lime to raise the pH, removing slimes and silts, ventilating the pipes, and increasing the rate of flow.

Corrosion of Metals

The corrosion of reinforcing steel is the major durability problem for concrete highway structures and parking garages. This is in spite of the fact that, except for certain exposure conditions, reinforcing steel does not normally corrode. However, when corrosion does occur, the formation of rust is an expansive reaction that leads to cracking and spalling of the concrete above the rusting steel. Corrosion of other metals may also be of importance and will be discussed briefly.

Mechanism of Corrosion Rusting is an electrochemical process that requires a flow of electrical current for the chemical corrosion reactions to proceed. For the typical case of the corrosion of iron, the principal element in reinforcing steel, oxygen and moisture must be available. At some locations on the reinforcing steel, the iron is oxidized; elsewhere, water is reduced by the oxygen to produce hydroxyl (OH^-) ions. Oxidation occurs at an *anode*, producing two electrons and a ferrous ion:

$$Fe \rightleftarrows 2e^- + Fe^{2+} \tag{18.13a}$$

The reduction reaction occurs at the *cathode*.

$$\tfrac{1}{2}O_2 + H_2O + 2e^- \rightleftarrows 2OH^- \tag{18.13b}$$

As shown in Figure 18.8 (for a reinforcing bar in concrete), the process requires the movement of electrons through the metal between the anode and the cathode and the ability of OH^- ions to migrate from the cathode to the anode. The electrical current within the metal exactly balances the ionic current through the concrete.

At the anode, hydrous iron oxides form, usually the first of which is ferrous hydroxide:

$$Fe^{2+} + 2(OH)^- \rightarrow Fe(OH)_2 \tag{18.14a}$$

The spontaneous oxidation of ferrous oxide to hydrated ferric oxide (rust) occurs rapidly:

$$2Fe(OH)_2 \xrightarrow{O_2,\ H_2O} 2Fe(OH)_3 \rightarrow Fe_2O_3 \cdot nH_2O \tag{18.14b}$$

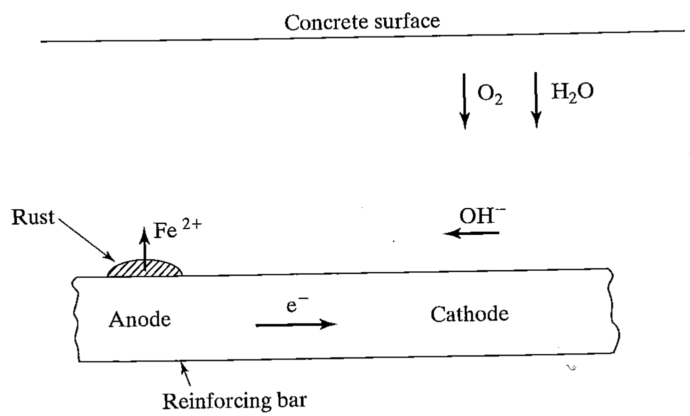


FIGURE 18.8
Corrosion of iron in concrete.

Ferric rust is responsible for the large volume increases that cause cracking and spalling. When oxygen is limited, such as in local regions that are fully saturated, corrosion may stop at Eq. (18.14a), forming "green" or "black" rust. In such a case, a bar can be completely corroded without cracking the concrete cover.

Different areas on a reinforcing bar may develop "active sites" with different *electrochemical potentials* (i.e., different tendencies for oxidation) and, thus, set up anode–cathode pairs (*galvanic couples*), as illustrated in Figure 18.8. Corrosion occurs at localized anodic areas. The development of anodic and cathodic areas can be caused by a variety of conditions, such as different impurity levels in the iron, different amounts of residual strain, or different concentrations of oxygen or electrolyte in contact with the metal. Figure 18.8 shows what is known as microcell corrosion, with anodic and cathodic regions on the same piece of metal. In many concrete structures, such as bridge decks, macrocells may develop, as differences in environment can cause entire layers of steel to become anodic (top mat) or cathodic (bottom mat), greatly increasing the rate of corrosion.

Different metals vary in their tendency to corrode. Some, like zinc, magnesium, and aluminum, are more active than iron; while others, such as chromium and lead, are less active. If two metals are connected, the more active metal will serve as the anode and supply electrons, thus protecting the less active metal (cathode) from corroding. Limiting access of moisture or oxygen will inhibit the corrosion reaction by limiting the formation of hydroxyl ions at the cathode.

The tendency of a metal to corrode under given conditions, referred to as the *half-cell potential*, is expressed in terms of a voltage measured with respect to a standard electrode such as a copper–copper sulfate electrode, CSE, or a saturated Calomel electrode, SCE. Such measurements can be made both in the laboratory and in the field and require electrical connection to the steel and saturation of the surrounding medium (in our case, concrete). The voltage differential between the steel and the standard electrode is referred to as the corrosion potential. It must be emphasized, however, that the corrosion potential by itself provides no information on the rate of corrosion, which depends on electron flow. For example, a high tendency to corrode may exist simultaneously with a low corrosion rate, if little oxygen is present. Corrosion potential

tests are widely used to evaluate the condition of structures such as reinforced concrete bridge decks.

Corrosion in Concrete Under normal conditions, with adequate concrete cover and in the absence of foreign ions, reinforcing steel does not corrode in concrete. The reason is that in the highly alkaline environment that exists in concrete (pH greater than 13 due to KOH and NaOH in the pore solution), the $Fe(OH)_2$ that forms at the anode is oxidized to γ-ferric hydroxide:

$$Fe(OH)_2 + O_2 \rightarrow \gamma\text{-}FeOOH + H_2O \tag{18.15}$$

γ-FeOOH provides a tightly adhering passive oxide film on the surface of the iron that limits access of oxygen and moisture to the metal and prevents corrosion. The protection is similar to that provided by the oxide coating that protects aluminum from corrosion in air. If the pH drops below 11.5, the passive iron oxide layer is destroyed, causing a normal, porous oxide layer (rust) to form during corrosion. This critical reduction in pH occurs when first the alkalies and then calcium hydroxide are converted to carbonates by atmospheric carbonation. In a well-cured concrete with a low *w/c* ratio, the depth of the carbonated zone is unlikely to exceed 25 mm (1 in.); therefore, concrete cover of 25 to 40 mm (1 to 1.6 in.) over reinforcing bars should provide adequate protection from corrosion in most instances. Where more severe conditions of exposure are encountered, the cover should be increased to at least 50 mm (2 in.). In concrete with high permeability, however, even 50 mm (2 in.) of cover may not be adequate. Penetration of CO_2 can be accelerated by the presence of microcracks in concrete due to the effects of external or internal stress. The rate of penetration in uncracked concrete follows Fick's Second Law [Eqs. (18.2) and (18.3)], with a depth of penetration that is proportional to the square root of time.

Even in fully carbonated concrete, where the reserve of alkalinity has been lost, corrosion of reinforcing steel may not be a problem, as long as the permeability of the concrete is low. The same applies for concretes made with calcium aluminate cements or pozzolanic admixtures, such as silica fume, slag, and fly ash, where the pH has been reduced due to use of the admixture. Under such conditions, the rate of corrosion depends on the availability of oxygen and moisture at the surface of the steel. Concrete with low permeability limits the rate of corrosion by limiting the diffusion of oxygen. [It is generally recognized that low *w/cm* ratio concretes containing silica fume exhibit superior corrosion performance to concretes with the same *w/cm* ratio, but without silica fume. The lower permeability more than makes up for the drop in pH.] If the concrete pores are filled with water, diffusion of oxygen will be further reduced. Thus, very low or very high moisture contents are beneficial to limiting corrosion, while intermediate moisture contents may be dangerous.

Invariably, reinforced concrete structures crack. The nature of the cracks can have a significant effect on corrosion performance. When load-induced cracks are perpendicular to the reinforcing bars (the typical condition), the width of cracks has relatively little impact on corrosion. Of course, the wider the crack, the greater the initial activity at the exposed anodic sites. Over time, however, the rate of corrosion is controlled by the great mass of reinforcing steel that is protected by concrete and serves as

the cathode. If the concrete cover is of adequate thickness and low permeability, diffusion of oxygen and water to the reinforcing bar is limited and the rate of corrosion is greatly diminished. Thus, the width of load-induced cracks, which generally ranges from 0.1 mm to 0.4 mm (0.004 to 0.016 in.), is not of particular concern. The story changes significantly, however, if the crack is parallel to the reinforcing bar, such as may occur due to settlement cracks in the upper layer of a concrete bridge deck. Since the settlement cracks may expose the full length of the bar to oxygen and moisture, the corrosion protection provided by the surrounding concrete may be severely diminished, especially in the presence of chloride ions.

Effect of Chloride Ions Chloride ions have the special ability to destroy the passive oxide film on steel, even at high alkalinities. The problem occurs due to the formation of a soluble iron–chloride complex that results in the deposition of loose porous rust and release of the chloride to continue the attack:

$$Fe^{2+} + Cl^- \rightarrow [FeCl\ complex]^+ \qquad (18.16a)$$

$$[FeCl]^+ + 2OH^- \rightarrow Fe(OH)_2 + Cl^- \qquad (18.16b)$$

In the absence of oxygen, chloride ions can also result in the formation of nonexpansive ferrous chloride corrosion products.

The amount of chloride required to initiate corrosion depends on the pH of the solution in contact with the steel (Figure 18.9), and comparatively small quantities are needed to offset the basicity of portland cement. Chlorides may enter concrete from three major sources: from $CaCl_2$ added as an accelerating admixture; from deicing salts used on pavements and bridge decks; and from seawater or salt spray. Prestressed elements should not be exposed to chloride without adequate protection since (1) the use of higher strength steel results in a smaller cross section which is, thus, more susceptible to the loss of a given weight of metal and (2) when steel is under tensile stress, corrosion may proceed more rapidly, owing to rupture of the passive film, and *stress corrosion* may occur. Thus, the chloride ion may accelerate corrosion at lower concentrations than usual (see Chapter 8).

The corrosion of reinforcing steel in bridge decks that are regularly treated with deicing salts is one of the most acute durability problems in North America. Deicing salts are generally mixtures of NaCl and $CaCl_2$, and much of the salt will penetrate into

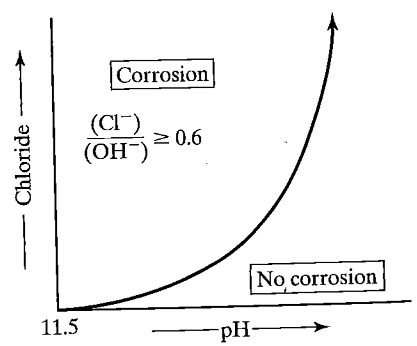


FIGURE 18.9

Effect of chloride and pH in the liquid phase on the corrosion of steel reinforcement. [From G. J. Verbeck in *Corrosion of Metals in Concrete*, SP-49, American Concrete Institute, Detroit, MI, pp. 21–26 (1975).]

the pores of the concrete and slowly diffuse down to the reinforcement. Of the two salts, $CaCl_2$ has the greater effect, since it reduces the pH of the pore solution, while NaCl has little effect on the pH. Little chloride is lost once it enters the concrete, so that there is a steady buildup of chloride ion until critical concentrations (0.6 to 1.2 kg Cl^-/m^3 or 1.0 to 2.0 lb Cl^-/yd^3 of concrete) are reached adjacent to the reinforcing bars. Corrosion of reinforcing steel in structures exposed to seawater or salt spray is another severe problem. It has been shown that the high concentration of chloride ion in seawater is such that it will penetrate even into high-quality concrete beyond the depth of cover normally specified. Whether severe corrosion will occur depends on other factors, such as the availability of oxygen at the surface of the reinforcement and the extent of other forms of deterioration that can take place in seawater.

Protection against Corrosion With some study of Eqs. (18.13a) and (18.13b) and Figure 18.8, it is clear that strategies to combat corrosion of reinforcing steel involve restricting the availability of oxygen and moisture and preventing electron flow from the anode to the cathode. With this in mind, the strategies can be divided into four categories: (1) reduction of permeability of the concrete, (2) protective membranes on the concrete, (3) protective coatings on the steel, and (4) suppression of the electrochemical process.

The use of high-quality, impermeable concrete with a low *w/c* ratio and adequate concrete cover can do much to alleviate the problem. [Adequate cover often means extra cover. Due to variations in the placement of reinforcing steel, a specified cover of 65 mm ($2\frac{1}{2}$ in.) is needed to provide a 90% probability that the minimum cover is not less than 50 mm (2 in.).] The addition of pozzolans, especially silica fume, and the use of high-density concrete overlays, with or without silica fume, significantly lower the permeability of concrete. Although chloride ions will continue to penetrate concrete regularly exposed to deicing salts or seawater, the rate of penetration will be considerably reduced. The rate of diffusion of oxygen will also be reduced, and its availability may limit the rate of corrosion, even when the critical chloride ion concentration has been reached. Furthermore, concrete of a low *w/c* ratio has better resistance to cracking and spalling because of its higher strength. Corrosion inhibitors of the organic type protect reinforcing steel principally by reducing the permeability of concrete.

Protective membranes have been used effectively on bridge decks subjected to deicing chemicals. The most effective membranes for bridge decks are placed directly on the newly constructed deck and then covered with a layer of asphalt concrete [50-mm (2-in.) minimum]. With proper drainage, the membranes ensure that deicing chemicals never come in contact with the upper surface of the concrete. Overlays using polymer concretes or polymer latex-modified concrete, although not widely used, have been effective. However, overlays of low slump–low *w/c* ratio concrete and overlays containing silica fume have been more cost effective, with the low *w/c* ratio overlays appearing to have the edge. Traditional surface treatments with water-repellant materials, such as linseed oil and kerosene and asphalt emulsions, are destroyed by traffic wear and weathering and provide little protection against corrosion. Penetrating sealants are not widely used for corrosion protection.

Based on experience, the most effective procedure for protecting reinforcing steel appears to be the application of fusion-bonded epoxy coatings to the surface of

the reinforcement. Where the coating is complete and adheres tightly to the steel, the epoxy prevents the formation of anodes and, perhaps even more importantly, prevents the steel from acting as a cathode to support corrosion at those locations where the coating may have been penetrated. With the widespread use of epoxy-coated reinforcing steel, starting in the late 1970s and early 1980s, the rate of deterioration of bridge decks in North America has been greatly reduced. In spite of its widespread success, epoxy-coated reinforcing steel has not been without problems. In well-publicized cases, improperly applied epoxy has been shown to accelerate corrosion as the result of a process known as crevice corrosion, which involves the concentration of chloride ions and, thus, highly acidic conditions in regions with very limited oxygen availability (i.e., under the coating). As a result, special emphasis is placed on obtaining tightly adhering coatings. The effectiveness of coatings also depends on how well they retain their integrity during fabrication (including bonding to the steel), transportation, handling during erection of the structure, and under the service environment. Even when properly applied, there is strong evidence that coatings that are used in saturated conditions become soft over time.

Concern with the limitations of epoxy-coated steel has led to ongoing efforts to improve the properties of the epoxy and to the development of alternate reinforcing bars, including solid stainless-steel bars and bars clad in stainless steel. Other cladding materials, such as copper and nickel, have been evaluated, but not pursued to any great length. Copper has been shown to retard the hydration reaction in the cement paste adjacent to the reinforcement. Zinc-coated (galvanized) reinforcement has been used with mixed success. The use of nonmetallic reinforcing bars (e.g., fiber-reinforced polymers) is also under study.

Suppression of the electrochemical corrosion of iron is the basis of *cathodic protection* and inorganic corrosion inhibitors. For cathodic protection, used most often as a corrective procedure, but at times in new construction, current is applied to reinforcing bars (which must be connected electrically) in the opposite direction to the current flow in spontaneous corrosion, using either an inert or sacrificial anode. If the current is sufficiently large, the iron is made cathodic and corrosion is prevented. [The use of galvanized steel, but not necessarily galvanized reinforcement, is based on the zinc coating not only cutting off oxygen and moisture to the steel, but serving as a sacrificial anode by corroding first and supplying electrons to the protected steel.] Sacrificial anodes, such as zinc or magnesium, can be placed in a concrete bridge deck or buried close to the structure. When an impressed current is used (direct current with an inert anode), the power requirements are surprisingly low. Cathodic protection has the added benefit of driving chloride ions away from the reinforcing steel and increasing the alkalinity in the region of the reinforcement. On the negative side, there is some evidence that the process increases the porosity of the paste in the vicinity of the bars, with a negative effect on bond strength between the steel and the concrete.

Various chemicals are known to inhibit corrosion of steel. Organic corrosion inhibitors (mentioned earlier) reduce the permeability of the concrete, limiting access of oxygen and moisture to reinforcement. Inorganic corrosion inhibitors help suppress the electrochemical process. The most widely used inorganic inhibitor, calcium nitrite (also an accelerator), converts ferrous oxide to γ-ferric hydroxide, which is redeposited on the surface to maintain the passivity of the steel.

$$Fe^{2+} + OH^- + NO_2^- \rightarrow NO\uparrow + \gamma\text{-FeOOH} \quad (18.17)$$

In the process, the nitrites are used up. Thus, for long-term protection, higher doses of calcium nitrite are needed, the higher the expected concentration of chlorides.

Corrosion of Other Embedded Metals Aluminum can be severely corroded by continued exposure to moist concrete, since its passive film is destroyed at high alkalinities. This results in the formation of aluminum hydroxide and hydrogen gas:

$$Al + 2OH^- + 2H_2O \rightarrow Al(OH)_4^- + H_2\uparrow \quad (18.18)$$

Corrosion of aluminum is aggravated in the presence of chloride ions (as is the case for steel) or if the aluminum is in contact with reinforcing steel (since it then acts as a sacrificial anode). Spalling of concrete and destruction of aluminum conduit are typical consequences. The chemical reaction shown in Eq. (18.18) is used to advantage in producing expansive grouts, since aluminum powder can be used to generate hydrogen gas, which will cause grouting materials to expand and fill narrow spaces, such as under the bases of machines (see Section 20.4). Like aluminum, lead and zinc will corrode in concrete, although the latter is more resistant to corrosion. In corrosion tests, copper-clad reinforcement has performed well, but has been observed to retard cement hydration in a region extending 0.25 to 0.50 mm (0.01 to 0.02 in.) from the bars into the surrounding concrete.

18.4 PHYSICAL ATTACK

Freezing and Thawing

Porous materials containing moisture are susceptible to damage under repeated cycles of freezing and thawing (frost attack). Hardened cement paste, which has a high porosity, is particularly susceptible to such conditions, and concrete may be destroyed in a single winter in northern climates. Fortunately, as discussed in Chapter 8, air entrainment has proven to be an effective and reliable means of protecting concrete from frost attack. It is of interest to examine the mechanism by which damage occurs on repeated freezing and thawing and thereby determine critical factors affecting frost resistance and the reasons why air entrainment is effective.

Freezing of Cement Paste When saturated concrete is cooled below 0°C, immediate freezing of most of the water in the cement paste does not occur. It must be remembered that paste contains a wide spectrum of pore sizes, and it can be shown thermodynamically that water in capillary pores will not freeze until the temperature is lowered below 0°C by an amount that depends on the diameter of the pore or, more accurately, the diameter of the neck of the pore. For example, water in pores of 10-nm diameter will not freeze until −5°C (23°F), and in pores of 3.5-nm diameter, water will not freeze until −20°C (−4°F). The presence of alkalies in pore water may result in a decrease in freezing temperature on the order of another 1°C. Even when the freezing temperature is reached, a significant portion of the water in concrete will supercool rather than freeze, since a seed or nucleus (ice crystal) is required to initiate the formation of ice. Seeds will form on the exterior of concrete and in large voids within the concrete, penetrating into the body of the cement paste as the temperature drops.

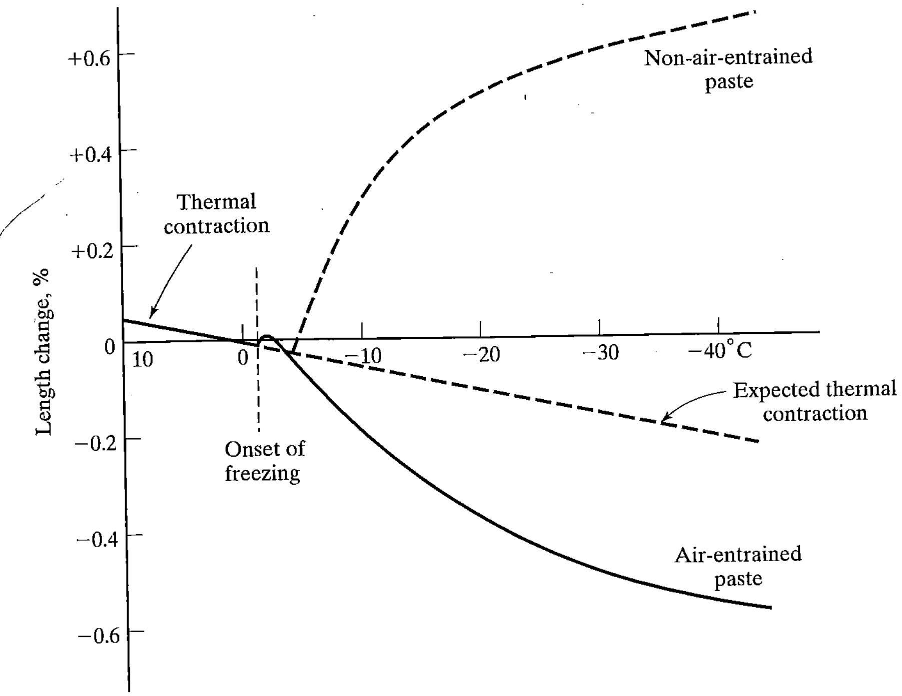


FIGURE 18.10

Volume changes occurring in cement pastes as the temperature is lowered. [Adapted from T. C. Powers and R. A. Helmuth, "Theory of Volume Changes in Hardened Portland Cement Paste During Freezing," *Proceedings of the Highway Research Board*, Thirty-Second Annual Meeting, pp. 285–297 (1953). Copyright by the Highway Research Board, National Research Council, Washington, DC.]

Crystals do not easily penetrate small capillary pores and can do so only when the temperature drops well below 0°C. Finally, water adsorbed on the surfaces of C–S–H, which forms on the surfaces of capillary pores and also creates micropores within the paste, will never freeze, although it may migrate to the capillary pores, where ice has formed. It has been shown that in an unprotected paste, severe dilation accompanies freezing (Figure 18.10), which leads to internal tensile stresses and cracking. In an air-entrained paste, very little dilation occurs and considerable shrinkage during freezing is observed at low temperatures.

Mechanism of frost attack. Several different processes can contribute to paste behavior during freezing. These are generation of hydraulic pressure by ice formation, generation of osmotic pressure due to the increase in solute concentration in the pore water adjacent to freezing sites, desorption of water from C–S–H, and segregation of ice. A 9% volume increase occurs as water turns to ice, but this change is insufficient to account for all of the dilation observed in cement paste and in concrete. Some of the dilation that occurs on first freezing is probably due to direct expansion of ice in microcracks, and this will increase if progressive microcracking occurs during continued freezing and thawing.

However, the major dilation that occurs must be attributed to other processes within the cement paste. For many years, the major contributor to dilation was believed to be *hydraulic pressure*, as proposed by T. C. Powers. He explained that as ice forms in a capillary, the accompanying volume increase causes the residual water to be compressed, and this pressure can only be relieved if the water can escape from the capillary to a free space by diffusing through unfrozen pores. However, if the water has too far to move to an escape boundary, the capillary will tend to dilate and the surrounding material will come under stress, eventually causing a tensile failure. As the temperature is progressively lowered, more capillary water is involved in freezing, increasing hydraulic pressures and thereby increasing microcracking and dilation.

Later, Powers demonstrated that hydraulic pressure could not be the major source of damage. He observed that when partially dry, non-air-entrained paste is frozen, it first shrinks and then expands, even if initially dry enough to provide an empty pore volume great enough to accept the 9% increase in volume. The expansion continues indefinitely even if the temperature is held constant, while under the same conditions, air-entrained cement paste shrinks indefinitely. Furthermore, freezing damage is observed with liquids that do not expand upon freezing. These observations indicate that the water is moving *towards*, rather than away from, the freezing sites. There are two possible explanations for this behavior. First, pore ice nucleates from the pore solution, resulting in a steadily increasing concentration of solute in the liquid adjacent to the freezing sites. This higher concentration will spontaneously draw water from the more dilute pore solution in the surrounding unfrozen paste through the process of *osmosis*. Therefore, once ice begins to form, the movement of water creates an *osmotic* pressure that can eventually cause the surrounding paste to crack. Second, the chemical potential of water in the form of ice is lower than that of the supercooled water in unfrozen pores and is manifested as a lower vapor pressure. As a result, the effective relative humidity at the freezing sites is lowered, and there will be spontaneous movement of water towards the freezing sites to maintain equilibrium. The process can be viewed as a spontaneous desorption of water from C–S–H, which causes the paste away from the freezing sites to shrink, even as significant dilation occurs at the freezing sites. The process is analogous to the formation of ice lenses in soils, although on a much finer length scale. Both osmotic pressure and the differential in vapor pressure have a similar effect.

In a saturated, non-air-entrained paste, free space is available on the exterior of the specimen and at a relatively small number of widely spaced air voids. Thus, the capillaries serve as the principal freezing sites, with the result that the cement paste will be severely damaged upon freezing. The inclusion of entrained air, however, provides empty space within the paste to which the excess water can move and freeze without damage. The air bubbles have much larger volumes than the capillaries within cement paste, and therefore any water in the air bubbles will begin to freeze at a temperature close to 0°C. The air bubbles are mostly empty, but may have a film of solution on their surfaces that is thick enough to freeze. The formation of ice in the air bubbles then allows the processes of osmosis and desorption, reducing the level of saturation in the adjacent cement paste, to proceed harmlessly. The bubbles act as "safety valves" drawing water from the cement paste and serving as a reservoir for both ice and concentrated pore solution. The spacing factor (see Chapter 8) determines the average distance the

water must travel to reach the free space. This distance must not be too great if osmotic pressure is to be relieved; hence the requirement of a critical spacing factor. Upon thawing, the water in the air bubbles returns to the cement paste because of the higher surface tension in the smaller capillaries and pores. In this way, cement paste will not suffer freeze–thaw damage as long as the moisture content remains constant. Even properly air-entrained concrete, however, will eventually be damaged if it becomes fully saturated, with the air bubbles filled with water.

Using the mechanisms described, it can be shown that the resistance to freezing and thawing depends on the permeability, the degree of saturation of the paste, the amount of freezable water, the rate of freezing, and the average maximum distance from any point in the paste to a free surface where ice can form safely. Partially dry concrete will not suffer freeze–thaw damage because the larger capillaries are empty and provide the necessary free space throughout the paste. Air entrainment is thus required for protection only for concrete that is frozen in near-saturated conditions. The entrained air bubbles do not readily fill with water except when the concrete is immersed for long periods of time. Once the air bubbles are filled with water, however, the concrete has a greater amount of freezable water and will become highly susceptible to frost damage.

The buildup of ice in particular parts of a structure on a macroscale may aggravate frost damage in concrete, although it is not the basic cause. The formation of *ice lenses*, as this phenomenon is called, occurs in soils and is responsible for frost heave. The development of ice lenses in concrete requires the presence of large voids, which could be microcracks or voids occurring under reinforcement or large aggregate particles due to segregation or poor compaction. An external supply of water that can readily migrate to these voids is also needed.

It can be shown that concrete must develop a certain strength to withstand frost damage (i.e., young concrete will be damaged by freezing even when air entrained). Furthermore, theory predicts that, below a *w/c* ratio of 0.36, air entrainment is not needed if the paste is completely hydrated, since there is little freezable water in the concrete. However, experimental studies demonstrate that a *w/c* ratio of 0.24 or less is needed before entrained air is no longer required (Section 19.3).

Freezing of Aggregates Certain rocks are also susceptible to damage on freezing and thawing and, if present in aggregates, may contribute to concrete damage. The concept of hydraulic pressure is appropriately applied to rocks since the pores are much larger than those in cement paste and are readily filled with water. There is, thus, a critical size of aggregate above which the aggregate is liable to fracture. This critical size is a measure of the maximum distance water must flow to reach the outside surface in order to relieve hydraulic pressure. The size depends on freezing rate, degree of saturation, permeability, and tensile strength (i.e., the same factors as for cement paste). For most rocks, the critical size is greater than the maximum size used for coarse aggregates. Most rocks (granites and high-quality limestones, for example) have very little porosity, and since their permeability may be low, they do not saturate readily in concrete. Therefore, they have internal voids available to relieve hydraulic pressures. Very porous rocks, such as sandstones or synthetic lightweight aggregates, are hard to saturate fully and are easily dried. When frozen, water can easily escape and a high degree

of saturation is not critical. The most critical situation is rocks containing fine pores, but a relatively high absorption (total porosity) combined with a low permeability. Once saturated, the critical size of the rocks can become less than the size present in the aggregate and fracture of the aggregate can occur. This can occur in some fine-grained rocks, such as cherts and shales, and causes pop-outs at the surface of the concrete. Rapid damage is most prevalent at the surface because the aggregate is more prone to saturation at this point, and freezing and thawing occurs more frequently. However, D-cracking (Figure 18.11) is a more serious consequence of freezing and thawing damage due to aggregates (principally limestones) in midwestern states. Deterioration usually starts at the lower parts of concrete slabs adjacent to joints, where moisture accumulates, and eventually progresses through the entire slab. When D-cracking is observed at the surface, deterioration is well advanced in the lower part of the slab, and this generally occurs within 10 to 15 years.

Even though aggregates with high absorptions may not be damaged by frost action, the water that is forced from the aggregate can cause severe damage to the surrounding cement paste, which has significantly less permeability than the aggregate itself. Entrained air bubbles do not provide enough volume to protect the cement paste from the hydraulic pressure created at the paste–aggregate interface. Thus, to withstand cycles of freezing and thawing, concrete must consist of both properly air-entrained paste and durable aggregate.

Freezing and Thawing Tests A number of different tests have been developed to assess the long-term frost resistance of concrete. These involve subjecting concrete to different freeze–thaw cycles and measuring the progressive internal damage by monitoring weight loss, length change (dilation), decrease in strength, or dynamic modulus of elasticity. Tests differ in the nature of the freeze–thaw cycle used and the condition of the specimen during the test, but all tests are arbitrary since they involve only one set of conditions. As we have seen, the potential for frost damage depends on freezing rate, *w/c* ratio, time of moist curing, and degree of saturation, and these may vary widely in actual service conditions. Thus, predictions from freeze–thaw tests are

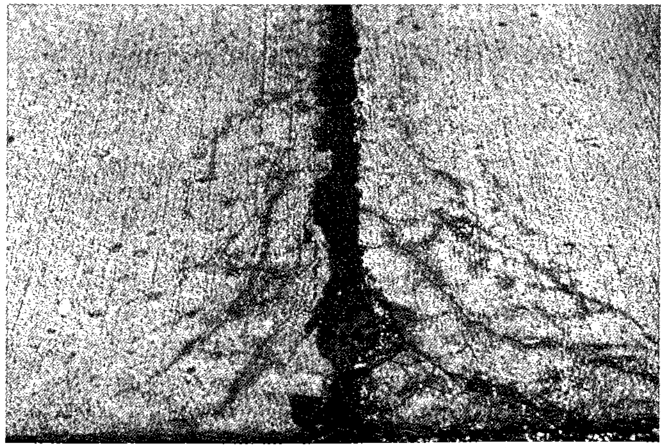


FIGURE 18.11

D-cracking in a concrete pavement. (Photograph courtesy of Portland Cement Association.)

not always a good indication of actual service performance, and results should be treated with caution.

The standard method of test described in ASTM C 666 prescribes a cooling and heating cycle between 4.4 and −17.8°C (40 and 0°F) to be completed in two to five hours. Freezing can be done while the specimen is immersed in water (Procedure A) or in air (Procedure B), but it is thawed in water in both cases. The test requires that not less than 25% and 20% of the cycle time be used for thawing in Procedures A and B, respectively. The test is continued (either continuously or intermittently) for 300 cycles or until the dynamic modulus has reached 60% of its initial value, whichever occurs first. A durability factor DF can then be calculated as

$$\mathrm{DF} = \frac{P \times N}{300} \tag{18.19}$$

where P is the percentage of the initial dynamic modulus after N cycles. There are no definite values of durability factor for acceptance or rejection of the concrete subjected to this test. Using Procedure A, value of less than 40 suggests that the concrete may be unsatisfactory, whereas above 60 it is likely to perform well, but there is no guarantee that this prediction will hold true. Both Procedures A and B are used by various agencies to evaluate the durability of coarse aggregate, as well as the durability of concrete mixtures as a whole. Procedure A is a especially severe since most specimens eventually become fully saturated and eventually fail. Even concretes that perform well in the field may perform poorly when subjected to cycles of freezing and thawing in water.

The major objection to ASTM C 666 is the unrealistically high rate of freezing, since in the field, rates exceeding 3°C/h (5°F/h) are rarely encountered. ASTM C 671 overcomes this objection by specifying one freeze–thaw cycle every two weeks, employing this rate of freezing with a rapid thawing regime. Damage is monitored by measuring linear expansion during the test, which is continued until critical dilation (i.e., until dilation begins to increase sharply) or a desired number of cycles is completed. This test allows the user to specify the curing history of the specimen and the exposure conditions to most nearly match the expected service conditions. ASTM C 682 for freezing and thawing of aggregates was mentioned in Chapter 7. This test involves casting concrete cylinders made with the aggregate in question and an air content matching the requirements for severe exposure in Table 10.2 (Section 10.3). The cylinders are tested and the results analyzed in accordance with ASTM C 671.

Salt Scaling

Concrete that is adequately air entrained for frost resistance may nonetheless be damaged by repeated reapplication of deicing salts. We have already discussed damage that can arise from accelerated corrosion of reinforcing steel caused by the chloride salts commonly used, but even properly air-entrained, high-quality concrete may be damaged directly by deicing chemicals that contain no chloride. Concrete that has suffered salt scaling becomes roughened and pitted due to the spalling of small pieces of mortar.

Mechanism of Scaling The exact causes underlying salt scaling are not known, but they probably involve more than one process. Salt solutions have a lower vapor pressure than pure water. The result is a lower rate of evaporation and a higher degree of saturation than is the case for concretes without deicers. It has been suggested that the consumption of heat required to melt ice when a deicer is applied causes a rapid drop in temperature of the concrete just below the surface, which may cause damage either from the effects of rapid freezing or stress caused by differential thermal strains. In either case, the additional free moisture present at the surface of the concrete may encourage the growth of microscopic or macroscopic ice lenses. The addition of salt to the alkaline pore solution will increase the effects of osmotic pressure. At low concentrations of salt, scaling appears to increase with increasing concentration, but then decreases at higher concentrations. Thermodynamic arguments are used to explain the drop-off in scaling. Laboratory tests indicate that concretes made with fly ash tend to have lower scaling resistance than other concretes, although such observations are rarely duplicated in the field.

Prevention of Scaling The use of an adequately air-entrained, low *w/c* ratio, low permeability concrete is the best protection against salt scaling. The air bubbles help relieve the buildup of pressure due to osmosis and the difference in vapor pressure over liquid water and ice, and low *w/c* ratio and low permeability limit penetration of water and deicers, while providing a material that is more able to withstand the stresses that cause salt scaling.

Scaling is most likely to occur on surfaces that have been overvibrated, troweled too early and too long, subjected to plastic shrinkage, or where excessive bleeding has occurred. Such surfaces tend to have a weak layer of paste or mortar either at the surface or just below and may have microcracks or bleeding channels that can transport surface solutions to lower levels. Careful attention to mix design, placing, and finishing should eliminate many potential problems. If adequate moist curing is following by a period of drying before deicing chemicals are applied, scaling resistance will be improved. ASTM C 672 is used for assessing scaling resistance of concrete surfaces. A solution of $CaCl_2$ is ponded on the specimens, which are then subjected to freeze–thaw cycles. (The freeze–thaw cycles are not the same as in the freeze–thaw durability tests.) Visual examination of the surface is made every five cycles.

18.5 CRACKING IN CONCRETE

Throughout this book, we have continually referred to potential cracking of concrete. Cracks may be caused by many different situations and may range from very small internal microcracks that occur on the application of modest amounts of stress to quite large cracks caused by undesirable interactions with the environment, poor construction practices, or errors in structural design and detailing. In extreme cases, the structural integrity of the concrete may be seriously affected. In many other instances, however, cracks do not affect the ability of concrete to carry load, but may affect the durability of the concrete by providing points of easy access to the body of the concrete for aggressive agents that might otherwise not seriously affect the material. In this section, we briefly review the causes of cracking and discuss methods of control.

TABLE 18.5 Causes of Cracking in Concrete Due to Interaction with Surroundings

Component	*Type*	*Cause of Distress*	*Environmental Factor(s)*	*Variables to Control*
Cement	Unsoundness	Volume expansion	Moisture	Free lime and magnesia
	Temperature cracking	Thermal stress	Temperature	Heat of hydration, rate of cooling
Aggregate	Alkali–silica reaction	Volume expansion	Supply of moisture	Alkali in cement, composition of aggregate
	D-cracking	Hydraulic pressure	Freezing and thawing	Absorption of aggregate, maximum size of aggregate
Cement paste	Plastic shrinkage	Moisture loss	Wind, temperature, relative humidity	Temperature of concrete, protection of surfaces
	Drying shrinkage	Moisture loss	Relative humidity	Mix design, rate of drying
	Sulfate attack	Volume expansion	Sulfate ions	Mix design, cement type, admixtures
	Thermal expansion	Volume expansion	Temperature change	Temperature rise, rate of change
Concrete	Settlement	Consolidation of plastic concrete around reinforcement		Concrete slump, cover, bar diameter
Reinforcement	Electro-chemical corrosion	Volume expansion	Oxygen, moisture	Cover, permeability of concrete

Causes of Cracking

Table 18.5 summarizes the kinds of cracking that can occur due to interactions involving the materials of concrete and its surroundings. In most instances, cracking originates internally, forming a network of microcracks throughout the concrete. Internal damage may be considerable before cracks are visible at the exterior surfaces. In other cases, such as humidity and temperature changes, localized large cracks may occur in the structure. Cracking may be used to help determine the cause of deterioration of concrete, since in many cases characteristic cracking patterns are produced (see Table 18.6). Concrete that will resist cracking under normal environmental conditions cannot be assured of remaining intact under catastrophic conditions, such as fire.

TABLE 18.6 Types of Cracking in Concrete Structures

Nature of Crack	*Cause of Cracking*	*Remarks*
Large, irregular, frequently with height differential	Inadequate support, overloading	Slabs on ground, structural concrete
Large, regularly spaced	Shrinkage cracking, thermal cracking	Slabs on ground, structural concrete, mass concrete
Coarse, irregular "map cracking"	Alkali–silica reaction	Extrusion of gel
Fine, irregular "map cracking" (crazing)	Excessive bleeding, plastic shrinkage	Finishing too early, excessive troweling
Fine cracks roughly parallel to each other on surface of slab	Plastic shrinkage	Perpendicular to direction of wind
Cracks parallel to sides of slabs adjacent to joints (D-cracking)	Excessive moisture contents, porous aggregates	Deterioration of concrete slab due to destruction of aggregates by frost
Cracks above and parallel to reinforcing bars	Settlement cracking	Structural slabs due to consolidation of plastic concrete around reinforcing bars near upper surface
Cracking along reinforcing bar placements, frequently with rust staining	Corrosion of reinforcement	Aggravated by the presence of chlorides

Control of Cracking

Cracking is best controlled during the design and construction phases. In many instances, cracking may be avoided by proper selection of materials, provided that the potential problem has been anticipated through a careful assessment of the expected environment. For example, unsoundness of cement should never be encountered when ASTM C 150 is adhered to, and proper testing of groundwater should enable severe sulfate attack to be avoided through the choice of an appropriate binder combination and *w/c* ratio.

Chemical attack of concrete involves ingress of moisture, either as a carrier for aggressive agents or as a participant in destructive reactions. Thus, precautions in mix design and construction practices that prevent the entry of water into and passage through concrete should improve durability. Concrete of low permeability can be assured by the use of sufficient (but not excessive) quantities of cementitious materials and low *w/cm* ratios, proper placement, consolidation and finishing, and adequate moist curing. Provision for effective drainage and the use of watertight construction joints can be very beneficial in some situations.

The use of surface sealing compounds can be of value in severe environments if they are properly applied. These can be of two types:

1. Materials that are applied to the surface to provide a waterproof, impervious coating.
2. Materials that penetrate the surface of the concrete and block the capillary pores.

The first type is exemplified by paints, but is more likely to be thicker, more robust coatings or overlays based on asphalt, epoxy resin, synthetic rubber, or other polymeric materials. Such coatings may also be used to keep moisture in and reduce drying shrinkage. Coatings may themselves contain cement and aggregate, examples being latex-modified mortars, polymer concretes, and asphaltic concrete. Materials that penetrate and block the capillary pores can be used to seal the surface of good-quality concrete, but they are not effective with poor-quality concrete.

Two types of cracking are often observed in plastic concrete. As discussed in Section 11.5, plastic shrinkage cracks occur when moisture is removed more rapidly from the surface of fresh concrete than it is replaced by bleedwater from below. When faced with high temperatures, high winds, and low relative humidity, construction procedures and protection for the surface of a slab must be adjusted to limit the potential for rapid evaporation. Settlement cracks, which occur above and parallel to reinforcing bars near the surface of concrete, can provide a ready path for corrosive chemicals to reach the reinforcement. The extent of settlement cracking will be decreased with reduced concrete slump, increased cover, and reduced reinforcing bar size.

Cracking due to drying shrinkage and thermal expansion is caused by tensile stresses that are created by differential strains that occur under nonuniform drying, temperature rise, or uneven restraint. Thus, shrinkage and thermal cracking resemble flexural cracking and can be controlled by suitable location of reinforcement, which will reduce the amount of cracking and will cause several fine cracks rather than a single large crack. The finer the crack, the less likely it is to contribute to durability problems. Crack widths less than 0.10 mm (0.004 in.) are desirable in cases where severe exposure is anticipated.

The use of properly located isolation and contraction joints helps reduce stress due to expansion and contraction of concrete. Rigid joints should be adequately designed to accommodate additional stress that may be caused by moisture or temperature changes. Shrinkage cracking in walls and slabs on ground usually cannot be completely avoided, necessitating the use of contraction joints. These are grooves cast into the concrete or sawn soon after hardening that provide planes of weakness where the crack will form preferentially. In this way, random cracking is avoided, and the cracks are located in a manner that allows them to be sealed against moisture. More contraction joints must be cut than are actually needed because the amount of restraint is not known. Cracked joints must be regularly maintained. For this reason, strategies to eliminate contraction joints in pavements have been sought; the use of expansive cements, continuously reinforced or prestressed pavements, or fiber reinforced concrete are possible solutions.

Many cracks in concrete structures are due to poor construction practices or errors in design and detailing. Examples of the former are the use of high *w/c* ratios and inadequate curing. An example of the latter is the use of reentrant corners in structural walls and slabs-on-grade (see Figure 18.12) without the provision of reinforcement to limit the width of the cracks or proper joint detailing to control crack location.

Cracking may be controlled if the magnitude or rate of environmental changes can be reduced. In this way, tensile stresses will be lowered and may be further reduced by tensile creep and, in the early life of a structure, by an increase in tensile strength

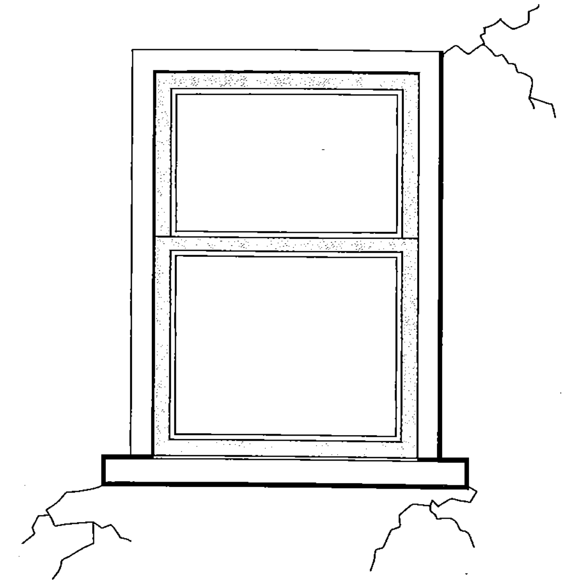


FIGURE 18.12

Typical crack patterns at reentrant corners. [From W. H. Price, *Concrete International*, Vol. 4, No. 1, pp. 40–43 (1982).]

(see Figure 18.13). Changes can be made by temporary protection of concrete or the use of reflective or impermeable coatings.

18.6 REPAIR AND MAINTENANCE OF CONCRETE

Once cracking or other manifestations of deterioration are visible, the concrete is more susceptible to damage, which may eventually render it unsuitable for further use. Although it is obvious that economic and political considerations will be important in decisions on maintenance and repair, from a technical point of view, early maintenance is desirable for maintaining the integrity of concrete. Preventative maintenance (regular inspection and restoration of sealed joints, drainage systems, etc.) will play an important role in the durability of concrete structures. Early attention to sealing of cracks and restoration of waterproof joints may eliminate the need for more costly repairs later. In cases where extensive deterioration has occurred or earlier measures have not controlled a problem, an investigation should be made to determine the cause and steps needed to counteract the situation during repair operations.

Materials for Repair

Many kinds of materials are used to repair and maintain concrete. These range from low-viscosity polymers for sealing very fine cracks, very rapid-setting cements for repairs in the presence of flowing or seeping water, and special concretes for overlays to portland cement, mortar, and concrete itself. A selection of materials is given in Table 18.7, but this can be used only as a guide since the diversity of materials is so great. The engineer will be faced with an array of potential materials to choose from, requiring a specialized knowledge for proper evaluation. A final selection will depend on many

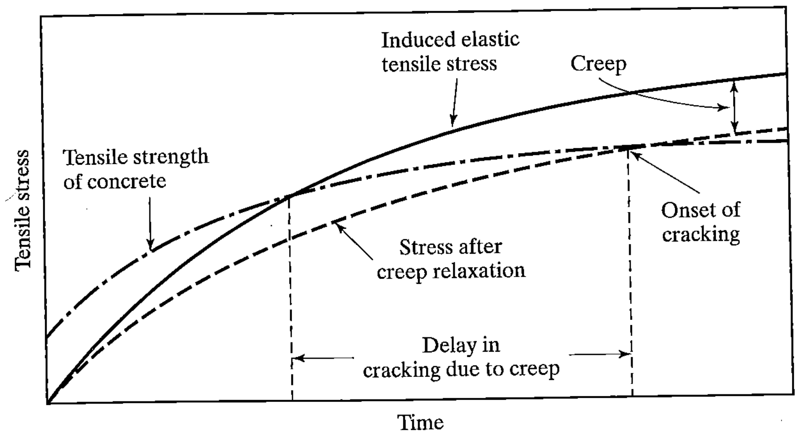


FIGURE 18.13

Relief of tensile stresses in concrete by creep. [Adapted from A. M. Neville, Properties of Concrete, 4th ed., John Wiley & Sons, Inc., New York (1996).]

TABLE 18.7 Materials for Repair of Concrete

Repair Operation	*Material*	*Comments*
Sealing of fine cracks	Epoxy resins	Good bonding properties even in the presence of moisture
Sealing of large cracks and joints	Portland cement mortar	Well compacted
	Cementitious grouts	
	Polymer mortar	Good bonding properties
	Putties and caulks	Based on synthetic polymers and tars
General sealing of surfaces	Synthetic polymers and asphalt coatings	
Localized patching of surfaces	Concretes or mortars using portland cement	
	Preplaced aggregate concrete	
	Rapid-setting cements	Calcium aluminate and regulated-set cements
	Polymer resins	Epoxies, polyesters; good bonding
Overlays and shotcreting	Portland cement concrete	Quick-setting admixtures
	Silica fume concrete	Low permeability
	Fiber reinforced concretes	Resistance to cracking
	Latex-modified concrete	Good bonding
	Polymer concretes	Good bonding
	Asphaltic concrete	
Strengthening	Additional internal or external reinforcing steel	
	Ferrocement	
	Fiber reinforced polymers	

factors, such as properties during repair, mechanical response, long-term durability, cost, and prior field experience.

Techniques of Repair

It is not possible in this book to provide more than a general discussion of the varied techniques that can be used in the repair and restoration of concrete. Nevertheless, there are three considerations that should be kept in mind. These are that efficient, durable materials depend on (1) use of a repair procedure that takes into account the cause of the damage, (2) proper selection of materials, and (3) proper preparation of the damaged concrete prior to restoration. The first consideration may depend on structural or material performance, or both, and, depending on the nature of the problem, require detailed engineering analysis and design. Selection of materials is discussed in the previous section, while preparation of the damaged concrete will be discussed in the ensuing sections. The following general points should be remembered:

1. All damaged materials should be removed until a sound surface is reached.
2. Where possible, the cavity should be prepared to ensure good bonding between the concrete and the repair material and to ensure proper consolidation.
3. Measures should be taken to remove aggressive materials or prevent their reentry and to select a repair material that is compatible with the structure and the type of damage.

Repair of Cracks Fine cracks, as small as 0.05 mm (0.002 in.), have been successfully sealed by the injection of low-viscosity polymeric grouts. Such materials should be capable of forming a solid polymer in situ after injection. Epoxies are a popular choice, and many proprietary formulations are commercially available. The epoxy is injected under pressure to penetrate the very fine and tortuous crack pattern that may exist. The success of pressure grouting depends on proper application to ensure that all cracks are sealed. Epoxy grouting can restore structural integrity as well as seal cracks against seepage. However, if the crack is stress induced, it is likely that, unless the condition that caused the cracking is corrected, new cracks will form. Epoxy has been used under water or in the presence of seeping moisture with good success, but the choice of suitable materials is important.

Larger cracks and joints may be sealed using cementitious mortars, grouts, and caulks or putties. For a durable and successful seal, the crack should be cleaned and cut back to form a V-shaped groove (in a process called routing) into which the sealant can be well compacted. A good quality portland cement mortar is satisfactory (and cheap) for larger cracks. In the presence of moisture, quick-setting admixtures should be used or the portland cement replaced by a quick-setting proprietary cement (inorganic or organic). For finer openings under dry conditions, caulks and putties based on organic polymers can be used. Sealants are of many types and properties; the selection should depend on considerations of anticipated service conditions, such as applied loads and conditions of exposure. Once cracks and joints are repaired, a general protective coating is both beneficial and often aesthetically pleasing. In cases where little seepage is likely to occur, a general protective coating may be sufficient.

Localized patching This may involve filling tie holes, bolt holes, prestressing ducts, and so on. The simplest approach is to use dry-pack mortar for shallow holes and conventional "replacement mortar" for deeper cavities or for filling around reinforcing bars. Pressure grouting may be required for deep and narrow cavities, such as prestressing ducts. Mortar should be as dry as possible, consistent with good compaction and pumpability. The use of admixtures to improve flow characteristics and to avoid shrinkage on subsequent drying may be advisable where the creation and maintenance of a good bond is important. For large cavities, replacement concrete may be used for economy; in difficult situations, the use of preplaced aggregate concrete (as discussed in Section 11.3) may be advisable.

The use of replacement mortar and concrete, or equivalent materials, such as asphalt or polymer concrete, is common practice in localized repair of pavements and floors. All too often, repairs are not durable because insufficient attention is paid to initial preparation. All unsound, unbonded concrete should be removed, since a good bond is required between the old concrete and patching material. If reinforcement is corroded, the surface rust should be removed, and where possible, it is advisable to completely expose the outer layer of reinforcement to provide additional "interlocking." Where patching is used in regions of corroded steel, some problems may occur because of differences in corrosion potential on the reinforcing steel between regions of new "clean" concrete and surrounding concrete containing chlorides. Bond will be improved by roughening the surface, or shaping a hole to provide mechanical interlock, or both. Priming with cement mortar or a polymer bonding agent will help develop additional chemical bond between the old and new concrete. Alternatively, the use of materials such as polymer concrete or latex-modified concrete will in themselves give a good bond.

Overlays The foregoing considerations will apply equally well to general overlays of surfaces where extensive deterioration does not warrant localized patching. Resurfacing of pavements and bridge decks is a common application. If overlays are to be bonded directly to underlying sound concrete, a strong bond should be developed, and the overlay should match the underlying concrete in thermal properties or have good crack-resistant properties. If this is not the case, differential movements will cause cracking in the overlay, which will destroy its integrity and protection. In such circumstances, a nonbonded overlay should be used that can move independently of the base concrete.

On vertical surfaces, pneumatic application of concrete (shotcrete) is often used. This is particularly important where considerable structural damage has occurred and additional reinforcement is required or where a reasonably thick layer of new concrete is desirable. The application of thinner coatings (or stuccos) can be done by spraying or hand application. Conventional portland cement concrete, using special quick setting admixtures, is most commonly used, but the use of modified concretes, such as fiber reinforced, regulated-set cement or latex additions, has been advocated.

Strengthening Repairs often involve steps that will increase the tensile strength of concrete. This can be done through the use of additional internal reinforcement, which is placed by drilling and grouting, or external reinforcement, often in the form of prestressing steel. Ferrocement, consisting of closely spaced layers of small diameter wire mesh in conjunction with a cement mortar, can be used to increase tensile strength at the surface of the repaired member. Externally applied steel plates attached by adhesives and bolts and sheets of fiber-reinforced polymers are also used.

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PROBLEMS

18.1. What is the effect of permeability on durability?

18.2. What is the relationship between time and the depth of penetration of chemicals that diffuse into concrete?

18.3. What does the appearance of efflorescence on the surface of a structure imply?

18.4. Why is concrete highly porous, but relatively impermeable?

18.5. Describe the mechanism of sulfate attack on concrete.

18.6. What strategies can be used to prevent sulfate attack?

18.7. Why is seawater not as corrosive as groundwater with the same concentration of sulfate ions?

18.8. What type of damage can occur when concrete is cured at elevated temperatures, such as used at precast plants?

18.9. Why do acids attack concrete?

18.10. What conditions must be present for the corrosion of steel to occur?

18.11. What strategies could you adopt to minimize the corrosion of reinforcing steel in concrete bridge decks?

18.12. Why do deicing salts cause severe deterioration of concrete bridge decks?

18.13. Why is concrete damaged by repeated freezing and thawing cycles?

18.14. How does air entrainment minimize the effects of freezing and thawing?

18.15. Why are some aggregates susceptible to damage from repeated freezing and thawing?

18.16. What problems are caused by the cracking of concrete?

18.17. How might cracks be repaired?

CHAPTER 19

High-Strength Concrete

The definition of *high-strength concrete* continues to change as advances in concrete technology make it easier to achieve increasingly higher strengths using conventional construction practices. Thus, what might have been considered high strength at one time is now *normal strength*. In the 1920s, anything above 21 MPa (3000 lb/in.^{2}) was considered high-strength concrete. By the 1950s, 35 MPa (5000 lb/in.^{2}) represented high strength. In the late 1970s, 40 MPa (6000 lb/in.^{2}) concrete was looked upon as high strength, and more recently 60 MPa (8500 lb/in.^{2}) has been considered the lower boundary for high-strength concretes. In some major metropolitan areas, concretes with compressive strengths of 100 to 130 MPa (15,000 to 19,000 lb/in.^{2}) are available as a matter of routine. How much things have changed is illustrated by some of the pioneering basic research on high-strength concrete carried out in the early 1980s in which the concrete had strengths of only 60 to 70 MPa (8,500 to 10,000 lb/in.^{2}). The rapid increase in available concrete strength is due principally to the development of superplasticizers and the application of mineral admixtures, especially silica fume.

High-strength concrete represents only a small fraction of concrete production. However, it offers important engineering and economic advantages for structures that can incorporate the smaller sections obtainable with a higher strength material. The earliest commercial applications of high-strength concrete involved its use in columns for high-rise buildings. Increased compressive strength allowed major reductions in column cross sections, providing more usable floor area. So important was this advantage that the design of entire structures was changed from structural steel to reinforced concrete, although most of the structure (floor slabs and beams) consisted of normal-strength concrete; without the use of high-strength concrete, reinforced concrete construction would not have been considered in many of these applications. As time has progressed, high-strength concrete has seen a wider application, including its use in building shear and bearing walls, and in bridge girders, especially in long-span bridges where flange thickness is controlled by concrete strength. In the latter case, the reduction in dead load provides a strong economic advantage for use of the higher cost material. Similar economic advantages are not available for floor slabs and beams in buildings because the higher strength cannot be used to appreciably modify the dimensions of these members. For some applications, the higher modulus of elasticity,

rather than strength, provided by high-strength concrete is the principal material property used in design. And finally, high-strength concrete is used increasingly in structures subjected to extreme exposure conditions, such as the surfaces of highway bridges and parking structures. The higher durability (principally due to the lower permeability) that comes with the low *w/c* ratio needed to produce high-strength concrete is used to advantage, independent of the higher strength.

19.1 MATERIALS SELECTION AND PROPORTIONING

The procedures used for the design and production of high-strength concrete differ from those used for normal-strength concrete, due principally to the greater sensitivity of higher concrete strengths to the specific materials and construction methods used. Typically, the design of high-strength mixes requires an investigation of a wider range of both materials and mix proportions, entailing a greater number of trial batches than used for normal-strength concrete. Laboratory trial batches need to be run with the same materials as those to be used for the job concrete, and ultimately trial batches are required using the same batching and mixing equipment as used on the job. In North American practice, there is no single generally accepted method for proportioning high-strength concrete, although a guide, ACI 211.4R, has been developed for selecting the proportions of high-strength concrete containing portland cement and fly ash. However, in spite of the lack of a single design method, significant guidance does exist for developing high-strength concrete mixes.

High-strength concretes have much lower *w/c* ratios and total water contents than conventional concretes, often at much higher slumps. Since mineral admixtures are nearly always used in high-strength concretes, the water/cementitious material ratio (*w/cm*), rather than *w/c* ratio, should be used in mix design. Values of *w/cm* ratio in the range of 0.20 to 0.35 are commonly used, with water contents in the range of 125 to 135 kg/m^3 (210 to 230 lb/yd^3) compared to 180 to 200 kg/m^3 (300 to 340 lb/yd^3) for non-air-entrained normal-strength concrete. The production of low *w/cm* ratio mixes requires the use of superplasticizers that, in turn, allow for the production of concrete with a 180 to 230-mm (7 to 9-in.) slump.

Design Considerations

Like the design of normal-strength concrete, the design of high-strength concrete includes the consideration of the properties of both the plastic and the hardened materials. High-strength concretes are often designed for low permeability, for example, to provide corrosion protection for reinforcing steel by limiting access of chlorides, but quite naturally, compressive strength is usually the principal consideration. While ages of 7 or 28 days are used to determine if the specified strength has been attained, it is usual when working with high-strength concrete to establish strengths at later ages, such as 56 and 90 or 91 days. Since few engineering structures are completed in a 91-day period and rarely is the full compressive strength of the concrete required during construction, the additional strength obtained due to continued hydration can be used to advantage.

In the process of developing mix designs for high-strength concrete, consideration must be given to the rate of strength gain and to differences in concrete strengths

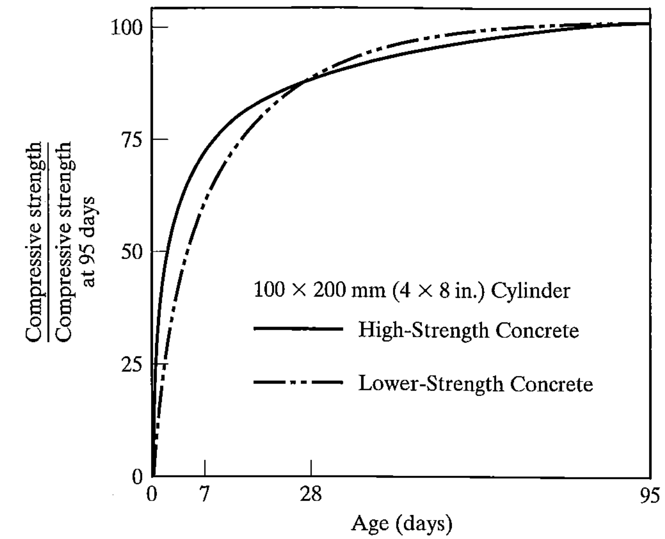


FIGURE 19.1

Normalized strength gain versus age for moist-cured high-strength and normal-strength concretes [From R. L. Carrasquillo, A. H. Nilson, and F. O. Slate, *ACI Journal*, Vol. 78, No. 3, pp. 171–178 (1981).]

obtained in the lab and in the field. High-strength concretes exhibit higher early strengths as a *percentage of long-term strength* than do conventional concretes, as shown in Figure 19.1. This higher rate of early strength gain is due to the fact that strength depends on both the degree of hydration of the cementitious materials and the density of packing obtained in the cement paste matrix, which is substantially greater for high-strength concrete due to both the lower *w/cm* ratio and the use of pozzolans, especially silica fume. In high-strength concrete mixes, a significant portion of the cement remains unhydrated; unhydrated cement contributes to early strength, but does not contribute to the continued gain in strength over time.

While it is clear that, at later ages, high-strength concrete exhibits a lower percentage increase in strength than normal-strength concrete, the absolute value of that increase in strength is generally higher for high-strength concrete. For example, concrete with a strength of 20 MPa (2900 lb/in.^{2}) at 7 days may have a strength of 35 MPa (5100 lb/in.^{2}) at 91 days. The 15 MPa (2200 lb/in.^{2}) difference represents a 75% increase in strength. In contrast, concrete with a strength of 64 MPa (9300 lb/in.^{2}) at 7 days might have a strength of 92 MPa (13,400 lb/in.^{2}) at 91 days. Although the gain in strength is only 44% compared with 75%, the absolute value of the gain, 28 MPa (4100 lb/in.^{2}), is considerably greater than that obtained by the normal-strength concrete. This higher absolute strength gain has led to modified recommendations for the selection of the required average strength of concrete, f'_{cr}, and acceptance criteria for concrete (see Chapter 15 and subsequent information in this chapter).

When high-strength concrete is used in the interior of a high-rise office building, durability is rarely a concern. There are many applications, however, where high-strength concrete will be exposed to the elements, raising concerns as to the ability of the material to withstand cycles of freezing and thawing or attack by aggressive chemicals. As discussed in Chapter 18, a reduction in the *w/cm* ratio will improve the durability of concrete. The use of a low *w/cm* ratio will also reduce the amount of freezable water. Nevertheless, it is the consensus that, unless the *w/cm* ratio is 0.24 or less, concrete must

contain entrained air to resist cycles of freezing and thawing and to prevent scaling damage. On the other hand, the addition of entrained air can be especially deleterious to strength for the rich mixes used for high-strength concrete. The reason is that, at the low *w/cm* ratios used for high-strength concrete, it is not possible to obtain large enough decreases in the water content to account for the strength reduction caused by the entrained air. However, the volume of entrained air required to provide protection is less than needed for normal-strength concrete.

As described in Chapter 13, increases in compressive strength do not result in commensurate increases in the tensile properties of concrete. Thus, the designer may want to use a larger maximum-size coarse aggregate and higher volumes of coarse aggregate to help increase the tensile and fracture properties of high-strength concrete, although in many cases, smaller coarse aggregate is needed to achieve high compressive strengths. Key considerations in the design of high-strength concrete mixes also include the workability of the concrete; its cohesiveness and ability to resist segregation; the time available for mixing, placing, and finishing; the techniques used for consolidation; and the effect of the heat of hydration.

Cement

The choice of cement requires greater attention for high-strength concrete than it does for normal-strength concrete because of the greater sensitivity of high-strength concrete to cement properties that are not evaluated or specified by the ASTM C 150, Specification for Portland Cement. Under ASTM C 150, cements are evaluated at *w/cm* ratios of about 0.5, without the use of superplasticizers, resulting in a cement paste matrix that is significantly different from that obtained at lower *w/cm* ratios with superplasticizers. Differences in cement properties that have relatively little effect on strength or rheology at high *w/cm* ratios can have a major effect on compressive strength at lower *w/cm* ratios, as illustrated in Figure 19.2. As a result, selecting an optimum cement invariably requires the use of a series of trials. Key properties of cement for use in high-strength concrete include C_3S, C_3A, and sulfate contents, as well as fineness. High C_3A contents limit flow characteristics at low water content. The sensitivity of high-strength concrete to the properties of the cement leads naturally to a need for uniformity in the properties of the brand selected. Variations in properties, reported under the provisions of ASTM C 917, should be limited to 4% for C_3S, 0.5% for ignition loss, 0.2% for sulfate content, and 375 cm^2/g for Blaine fineness.[1]

Although increases in cement fineness will increase the early rate of reaction, very high fineness is not desirable because it will increase the water demand and may have a negative effect on both the flow characteristics and the long-term strength that can be obtained for a given cement content. Similarly, the choice of a Type III cement may not be advantageous unless high early strengths are required.

When selecting cement for high-strength concrete, it is also important to determine if the cement and the admixture, usually a superplasticizer, are compatible. Compatibility can be established using fluidity tests prior to the evaluation of trial batches, but is often most easily determined using slump tests on trial batches of concrete.

[1]W. T. Hester, "High-Strength Air-Entrained Concrete," *Concrete Construction*, Vol. 22, No. 2, pp. 77–82 (1977).

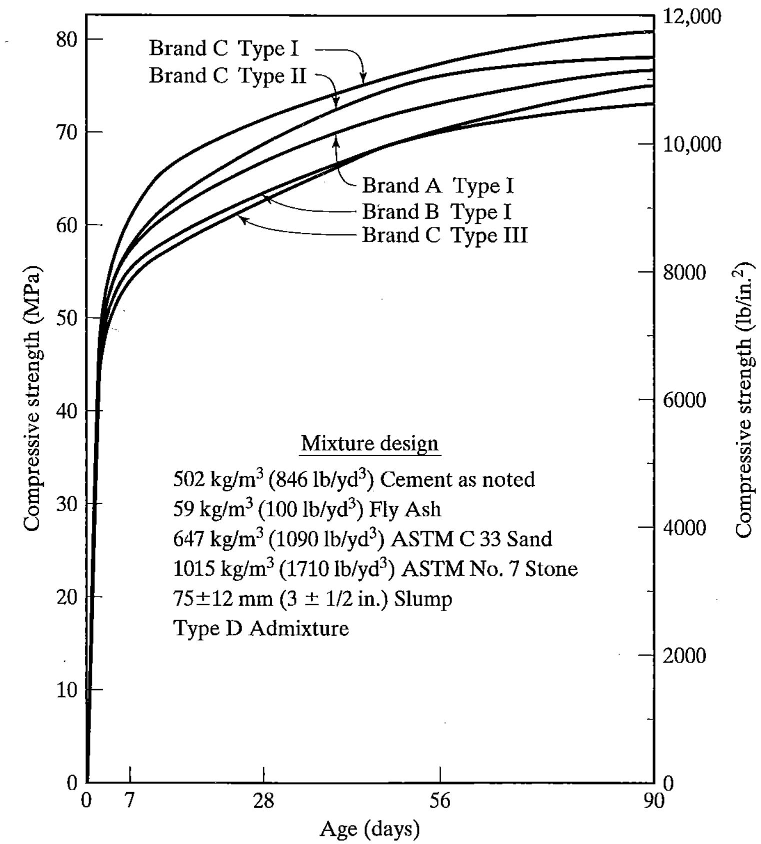


FIGURE 19.2

Effect of cement type and brand on the compressive strength of high-strength concrete [From *Task Force Report* No. 5, Chicago Committee on High-Rise Buildings, p. 9 (1977).]

As with normal-strength concrete, it is desirable to limit the cement content of high-strength concrete to the lowest value compatible with attaining the desired strength. For high-strength concrete, however, this consideration is more important since increasing cement to lower the *w/cm* ratio offers diminishing returns; the increased surface area obtained at high cement contents ultimately requires a higher water content, while the increased cement content itself results in the generation of additional heat of hydration. Typical cement contents for high-strength concrete range from 390 to 560 kg/m^3 (660 to 940 lb/yd^3). Values at the upper end of this range are not desirable, and concretes with cement contents greater than 600 kg/m^3 (1000 lb/yd^3) represent unduly expensive and impractical mixtures. Proper mix proportioning, including the use of superplasticizers and higher coarse aggregate contents than used for many normal-strength concretes, will result in the production of high-strength concretes with not only cement contents, but total cementitious material contents, in the lower half of the range.

Mineral Admixtures

Mineral admixtures can be used to replace a portion of the cement, since they contribute more to strength than an equal quantity of cement, especially at later ages. Fly ash, blast furnace slag, natural pozzolans, and silica fume are used as mineral admixtures in high-strength concrete. For the first three materials, the initial hydration reactions are slower than those that occur with portland cement alone, allowing workability to be maintained for longer periods of time. Fly ash usually, but not always, reduces the water demand and allows a reduction in the amount of superplasticizer needed to provide the desired level of workability compared to concrete without a mineral admixture or concrete containing silica fume. In contrast, the high surface area of silica fume particles increases water demand and, as a result, requires a greater amount of superplasticizer to maintain the desired slump. As shown in Table 19.1, the value of cement replaced depends on the particular mineral admixture, with values ranging as high as 65% (by mass) for blast furnace slag to as low as 5% for silica fume.

As described in Chapter 5, pozzolans add to the strength of cement paste by combining with calcium hydroxide produced by the hydration of calcium silicates in portland cement, and slags have cementitious properties of their own. The high fineness of silica fume also allows the material to act as a filler and to provide additional strength by reducing the void content in the cement paste matrix. As a result, high-strength concretes containing silica fume generally exhibit higher early strengths than do concretes without silica fume. Concretes containing the other three mineral admixtures exhibit slower rates of initial strength gain. Silica fume and slag or fly ash have been used together to produce improved short-term and long-term strengths for high-strength concretes. For example, ready-mixed concrete has been produced in Montreal with a one-year compressive strength of 137 MPa (19,900 lb/in.^{2}) [114 MPa (16,500 lb/in.^{2}) at 28 days and 126 MPa (18,300 lb/in.^{2}) at 91 days] with a total cementitious material content of 542 kg/m^3 (913 lb/yd^3) consisting of 60% blast furnace slag, 10% silica fume, and 30% portland cement.

Of the mineral admixtures, silica fume is most closely associated with producing high-strength concrete, since it generally produces the greatest strength increase for a given percent replacement of portland cement. Silica fume is often used to assist in developing concrete strengths as low as 60 to 65 MPa (8500 to 9500 lb/in.^{2}), but its inclusion appears to be *required* for concretes with strengths of 90 MPa (13,000 lb/in.^{2})

TABLE 19.1 Recommended Additions of Mineral Admixtures

Admixture	*Replacement Level (mass %)*	*Remarks*
Fly ash—Class F*	15–25	Ignition loss ≤ 3%
Fly ash—Class C*	20–35	
Ignition loss ≤ 3%		
Blast furnace slag	25–65	
Natural pozzolans	15–40	
Silica fume	5–15	Requires higher levels of superplasticizers

*ACI 211.4R.

and higher. Silica fume is also by far the most expensive of the mineral admixtures, with a cost of 7 to 10 times that of portland cement. Thus, a 10 to 15% replacement of cement by silica fume can double the cost of the cementitious material. In contrast, fly ash and blast furnace slag cost less than portland cement.

Aggregates

The properties that define high-quality aggregates for normal-strength concrete are also desirable for aggregates used to make high-strength concrete. However, unlike normal-strength concrete, which can be made with poorly graded and even weak aggregates using a somewhat higher cement content, high-strength concrete generally requires higher-quality aggregates. Since it is possible that the cement paste constituent in high-strength concrete will be stronger than many natural aggregates, aggregate strength becomes significantly more important as concrete strength increases. It is highly desirable that aggregate particles, especially the coarse particles, have no weak planes that would cause the aggregate to fail in a brittle manner as the concrete is loaded. To aid in workability, the aggregate particles should be as close to spherical or cubical in shape (see Figure 7.1, Section 7.1) as possible. It is generally desirable that the coarse aggregate be angular to improve paste–aggregate bond strength, but that the fine aggregate be rounded to aid in workability and limit the paste required to coat the smaller particles.

In addition to desirable mechanical and physical properties, aggregates used in high-strength concrete should be durable, so as to withstand potential problems with freezing and thawing, if the concrete will be exposed, and to be as resistant as possible to any potential chemical attack, since high-strength concrete is often used because of its higher durability rather than its higher strength. The lower permeability that comes with a reduced *w/cm* ratio should not be used as an excuse to use aggregates susceptible to the alkali–aggregate or alkali–carbonate reactions.

Fine Aggregate As a general rule, fine aggregate contents are lower for high-strength concretes than for normal-strength concretes due to both the higher cementitious material content and a higher coarse aggregate content compared to conventional mixes. Sand consisting of smooth, rounded particles, with quantities passing the 300-μm (No. 50) and 150-μm (No. 100) sieves at the lower end of the ASTM grading limits (Table 7.4, Section 7.1), are recommended to reduce the paste required to coat the fine aggregate particles. Because of the high cementitious material contents used in high-strength concretes, significant workability is provided by the cement paste constituent of the concrete. As a result, coarser sands are preferred for high-strength concretes, with values of fineness modulus in the range of 2.5 to 3.2, and values of 3.0 or greater generally recommended. Sands with values of fineness modulus below 2.5 are undesirable since they increase both paste and total water requirements and often produce concrete with a sticky consistency. For a given fineness modulus, the lower the void content, the higher the workability. The relationship between void content and workability ties in with the relationship between void content and angularity. A reduction in angularity leads to a reduction in void content and an increase in the lubricating capability of the fine aggregate.

TABLE 19.2 Volume of Coarse Aggregate per Unit Volume of Concrete for High-Strength Concrete Containing Portland Cement and Fly Ash[a]

	Volume of Dry-Rodded Coarse Aggregate per Unit Volume of Concrete for Sands with Fineness Moduli from 2.5 to 3.2			
Maximum Size of Aggregate, mm (in.)	9.5 ($\frac{3}{8}$)	12.5 ($\frac{1}{2}$)	19 ($\frac{3}{4}$)	25 (1)
Volume Fraction	0.65 (0.46)[b]	0.68 (0.55)[b]	0.72 (0.62)[b]	0.75 (0.67)[b]

[a]Adapted from ACI 211.4R. Reproduced with permission.
[b]Volume fraction for normal-strength concrete (F.M. = 2.80).

Coarse Aggregate In the development of mix designs for high-strength concrete, it is possible to take advantage of the extra lubrication provided by high cementitious material contents and superplasticizers by increasing the coarse aggregate contents above quantities that would be used for normal-strength concrete. The recommendations in ACI 211.4R are presented in Table 19.2. Like Table 10.8 (Section 10.3), Table 19.2 lists the recommended volume of dry-rodded coarse aggregate per unit volume of concrete. Values for coarse aggregates with maximum sizes ranging from 9.5 mm ($\frac{3}{8}$ in.) to 25 mm (1 in.) are included for sand with values of fineness modulus between 2.5 and 3.2 are included. The volumes are significantly higher than recommended for normal-strength concretes. For example, for 9.5-mm ($\frac{3}{8}$-in.) aggregate, the recommended fractional volume is 0.65 for high-strength concrete compared to 0.46 for normal-strength concrete for a fineness modulus of 2.80. For 25-mm (1-in.) aggregate, the respective fractional volumes are 0.75 and 0.67.

It is generally agreed that the compressive strength of high-strength concrete will decrease as the coarse aggregate size increases, as illustrated for a series of commercial mixes in Figure 19.3. According to ACI 211.4R, coarse aggregates with maximum sizes of 19 or 25 mm ($\frac{3}{4}$ or 1 in.) can be used for concretes with compressive strengths up to 60 MPa (9000 lb/in.^{2}); aggregates with maximum sizes of 9.5 or 12.5 mm ($\frac{3}{8}$ or $\frac{1}{2}$ in.) should be used for concretes with higher strength. However, additional experience indicates that higher compressive strengths [up to 100 MPa (15,000 lb/in.^{2})] can be obtained using the larger aggregates. As described in Section 19.3, an increase in total coarse aggregate content (more easily attainable with an increase in the maximum size of the aggregate) has the advantage of improving the tensile and fracture properties of concrete, as long as the aggregate is of high quality. Therefore, steps in mix design that enable the use of increased quantities of a larger coarse aggregate will provide benefits for structural applications in terms of higher tensile, shear, and bond strengths.

Chemical Admixtures

Chemical admixtures are essential for the production of high-strength concrete. Because of the high surface area of cement, it is impossible to attain workable mixtures with low *w/cm* ratios without the use of water-reducing admixtures or superplasticizers (see Chapter 8). High cement factors also increase the cost of concrete and increase

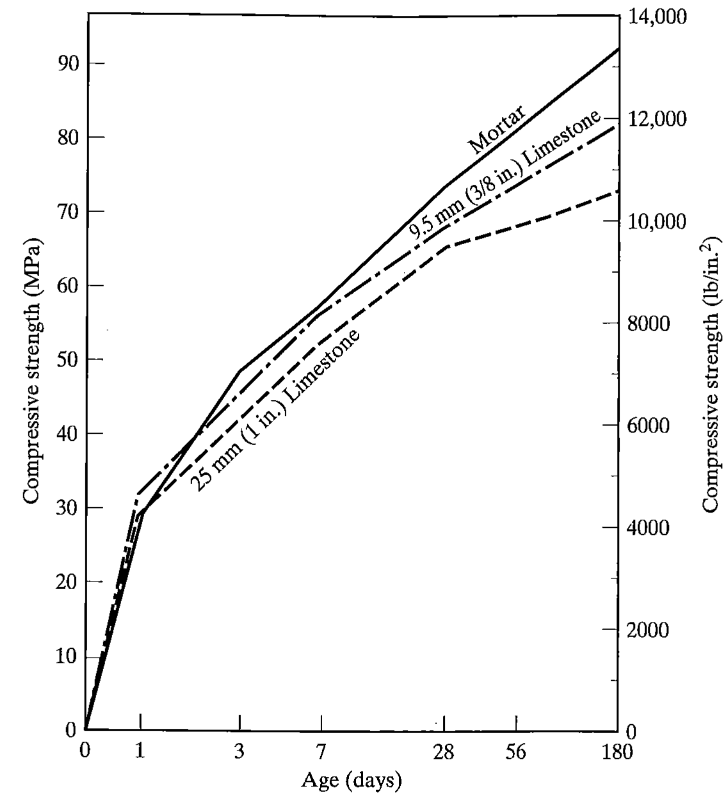


FIGURE 19.3

Compressive strength versus age for mortar and concretes with two sizes of coarse aggregate, *w/c* ratio = 0.32. [From J. E. Cook, *Concrete International*, Vol. 11, No. 10, pp. 67–75 (1989).]

problems related to the high heat of hydration. Thus, chemical admixtures are used to increase workability, decrease the *w/cm* ratio, and reduce cement contents.

In North American practice, high-strength concrete produced during the 1970s used combinations of water reducers and retarders to attain workable, low *w/cm* ratio mixes. Today, however, virtually all high-strength concretes use superplasticizers to attain the desired workability and *w/cm* ratio. These ASTM C 494 Type F admixtures are often used in combination with conventional water reducers (Type A), retarders (Type B), combination water reducers and retarders (Type D), and combination superplasticizers and retarders (Type G). In the mix design process, the water in the admixtures should be included in the calculation of the *w/c* or *w/cm* ratio. All superplasticizers do not behave in the same way, and the behavior of an individual superplasticizer alone or in combination with other admixtures may change depending on the dosage level (Figure 19.4) and the composition of the cement (especially C_3A, C_4AF, and sulfate). Accelerators (ASTM C 494 Type C admixtures) are not recommended for use in the manufacture of high-strength concrete.

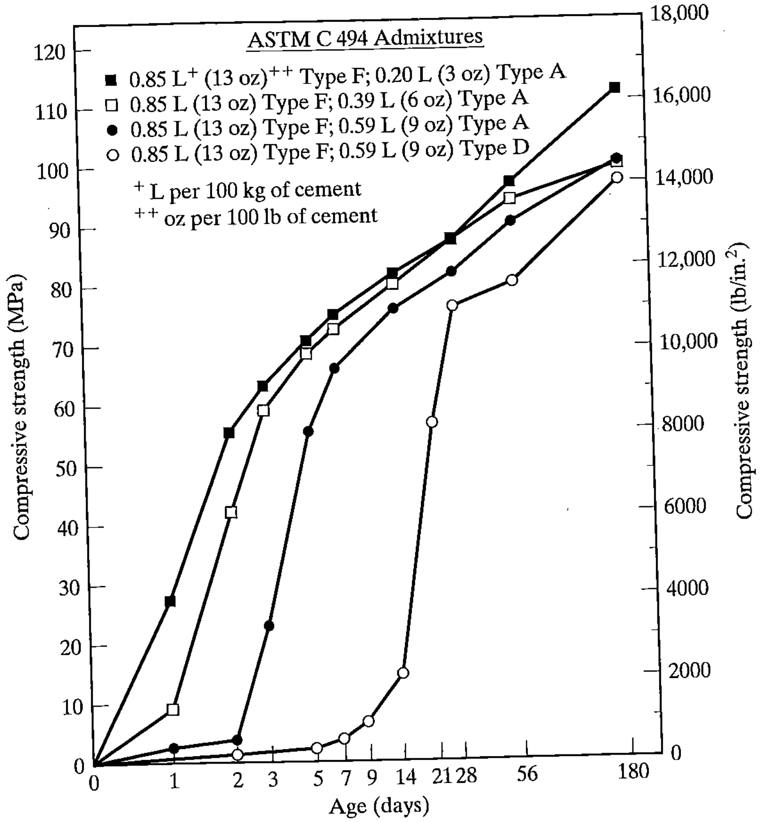


FIGURE 19.4

Effect of varying dosage rates of water-reducing admixtures and combination water-reducing and retarding admixtures on the strength of high-strength concrete containing a constant dosage rate of superplasticizer. [From J. E. Cook, *Concrete International*, Vol. 11, No. 10, pp. 67–75 (1989).]

In the mixing operation, a portion of the admixtures should be added as the concrete is batched to assist in initial mixing. The initial dose may consist of any or all of the admixtures used to reduce the water requirement or retard set or both. The balance of the admixtures is normally added at the job site. The total dosage rate used in high-strength concrete usually exceeds the manufacturer's "recommended rate." The use of a water reducer or superplasticizer during initial mixing is necessary to insure adequate mixing at the low *w/cm* ratio. The addition of superplasticizer at the job site not only helps gain the required workability for placement, but also results in a reduction of the amount of admixture required, since superplasticizers are more efficient when added after the cement has been wetted. The sensitivity of concrete properties to the combined cement–admixture system requires that trial batches be used to establish optimum dosage rates. If concrete workability is lost during the construction process, superplasticizers can be added to regain the lost workability. Such additions normally result in an increase in strength, as shown in Table 19.3.

TABLE 19.3 Effect of Redosing with Superplasticizer[a]

Total Admixture Dosage, L/m³ (oz/yd³)	*3.95 (102)*		*7.59 (196)*	
Age	*Compressive Strength, MPa (lb/in.²)*			
1 day	41.2	(5,980)	42.3	(6,130)
3 days	53.8	(7,800)	65.7	(9,530)
7 days	60.9	(8,830)	76.4	(11,080)
28 days	76.1	(11,040)	89.6	(12,990)
56 days	78.8	(11,420)	97.0	(14,070)
180 days	85.9	(12,450)	103.0	(14,930)

[a]Adapted from J. E. Cook, *Concrete International*, Vol. 11, No. 10, pp. 67–75 (1989).

Properties of Plastic Concrete

Plastic high-strength concrete looks and behaves differently from normal-strength concrete. Because of the high cementitious material content, the low *w/cm* ratio, and the use of superplasticizers, the cement paste constituent of plastic high-strength concrete has a generally sticky, honey-like consistency, resulting in a more cohesive mixture than is obtained for conventional concretes. This is true even at high slumps. When silica fume is used, the concrete exhibits very low bleeding, which often has a negative impact in the formation of plastic shrinkage cracks unless special precautions are taken to limit the rate of evaporation from the concrete surface. The cohesiveness of the mix is evident in the slump test in which, upon lifting the slump cone, the concrete is seen to slowly displace over a period of several seconds. The sticky consistency of the concrete can make the attainment of a smooth finish difficult.

For high-strength concrete containing portland cement and fly ash, ACI 211.4R recommends slumps of 25 to 50 mm (1 to 2 in.) prior to the addition of a superplasticizer and 50 to 100 mm (2 to 4 in.) if a superplasticizer will not be used. For many applications, however, water contents significantly below those needed to give measurable slumps are used, requiring the use of a water reducer or superplasticizer to allow initial mixing. While 50 to 100-mm (2 to 4-in.) slumps are often used for high-strength concretes, it is becoming more and more common to use 180 to 200-mm (7 to 8-in.) slumps when placing concretes with *w/cm* ratios in the range of 0.2 to 0.35. The higher slumps not only aid in consolidation, but also help ensure thorough mixing, which is critical in the production of high-strength concrete. Since most high-strength concrete uses a superplasticizer to attain the desired workability, mix design and construction procedures must take into account the potential for rapid slump loss. The rate of slump loss cannot be predicted for a particular mix, since this behavior depends on many factors. As a result, direct observation of the tendency for slump loss is required during laboratory and field testing to ensure that problems will not develop at the construction site.

When added at the correct level, superplasticizers do not increase segregation, but overdosing the mixture can result in severe segregation and retardation of the mix. These problems can usually be overcome by continuing the mixing process until the concrete begins to set, although this may require several hours.

TABLE 19.4 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregate for High-Strength Concrete Containing Portland Cement and Fly Ash.[a] Based on Sand with 35 Percent Voids.[b]

Slump		Water, kg/m^3 (lb/yd^3) of Concrete for Indicated Maximum Sizes of Coarse Aggregate							
mm	in.	9.5 mm	($\frac{3}{8}$ in.)	12.5 mm	($\frac{1}{2}$ in).	19 mm	($\frac{3}{4}$ in.)	25 mm	(1 in.)
30–50	1–2	185	(310)	175	(295)	170	(285)	165	(280)
50–80	2–3	190	(320)	185	(310)	175	(295)	170	(290)
80–100	3–4	195	(330)	190	(320)	180	(305)	180	(300)
Entrapped Air Content		3 (2.5)[c]		2.5 (2.0)[c]		2 (1.5)[c]		1.5 (1.0)[c]	

[a] Adapted from ACI 211.4R. Reproduced with permission.
[b] For void content (V) not equal to 35%, the quantity of mixing water should be adjusted by $5(V - 35)$ kg/m^3 or $8(V - 35)$ lb/yd^3.
[c] Mixtures containing a superplasticizer.

Water Content and Water/Cementitious Material Ratio

Water Content To achieve a high strength with a reasonable cement content requires both a low water–cementitious material ratio and a low water content. For concretes containing portland cement and fly ash, ACI 211.4R recommends the use of Table 19.4, which is analogous to Table 10.2 for normal-strength concrete. The water contents in Table 19.4 are 15 to 35 kg/m^3 (20 to 55 lb/yd^3) lower than those recommended in Table 10.2 for the same maximum size coarse aggregate and slump. Even so, the water contents in Table 19.4 are likely to produce mix proportions with excessively high cementitious material contents for concrete strengths in excess of 60 MPa (8500 $lb/in.^2$). For higher strengths, efficient mix designs are produced by significantly lowering the water content to values below 145 kg/m^3 (245 lb/yd^3), preferably in the range of 125 to 135 kg/m^3 (210 to 225 lb/yd^3).

Water–Cementitious Material Ratio The production of high-strength concrete requires, quite simply, the use of a low *w/cm* ratio. The value of the *w/cm* ratio depends on the desired strength, the age at which the desired strength is to be achieved, and the strength-producing properties of the particular combination of materials used in the concrete, including the cement–admixture combination and the aggregate quality, grading, and maximum size. Depending on the required strength, the *w/cm* ratio may range from 0.22 to 0.50. The need to run trial batches is emphasized by the scatter in strength versus *w/cm* ratio relationships, as shown in Figure 19.5. For example, the figure shows that, depending on the materials involved, 28-day compressive strength may range from 65 MPa (9500 $lb/in.^2$) to over 100 MPa (14,500 $lb/in.^2$) at a *w/cm* ratio of 0.30. Aitcin and Neville[2] point out that a *w/cm* ratio of about 0.22 represents the optimum for

[2] P.-C. Aitcin and A. Neville, *Concrete International*, Vol. 15, No. 1, pp. 21–26 (1993).

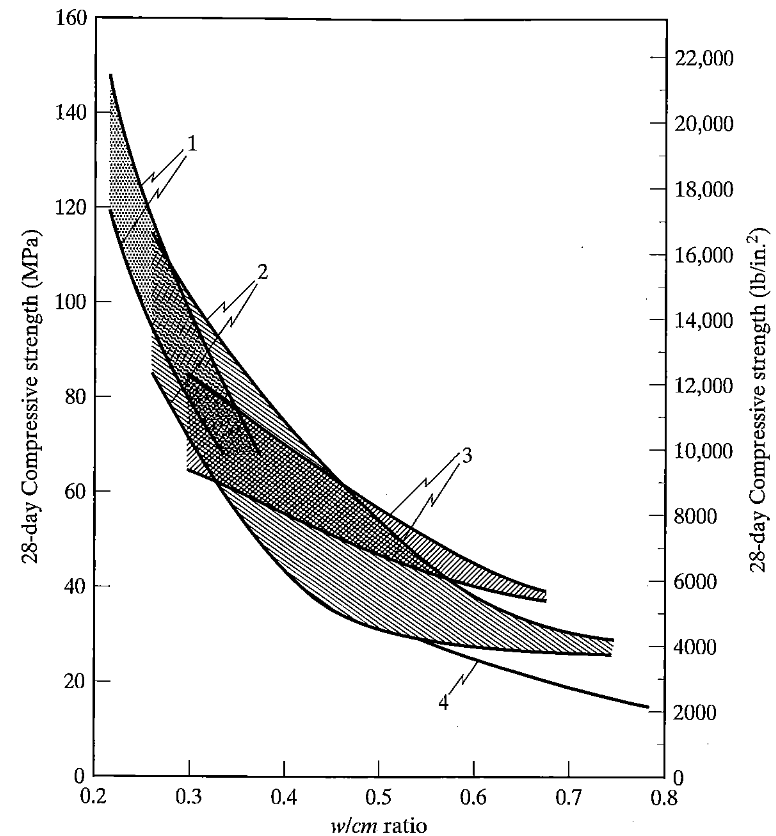


FIGURE 19.5

Compressive strength versus *w/cm* ratio for concretes from several sources. [Adapted from S. Mindess, in *High-Performance Concrete: Properties and Applications*, eds. S. P. Shah and S. H. Ahmad, McGraw–Hill, Inc., New York, p. 14 (1994).]

current cements and admixtures; above 0.22, the *w/c* ratio law governs, while below 0.22, a sufficiently dense cement-paste matrix cannot be obtained using current methods of field placement and consolidation. ACI 211.4R provides recommended maximum values of the *w/c* ratio for concretes containing superplasticizers (Table 19.5). Below *w/cm* = 0.4, full hydration of the cement will not occur because of space limitations, as described in Chapter 4. The concern, however, is to ensure maximum possible hydration to reduce capillary porosity. The dense microstructure that develops may limit the access of water to the interior and require longer moist curing to maximize hydration.

Air Entrainment

As described earlier, the current consensus is that entrained air is required for mixes with *w/cm* ratios greater than 0.24, if the concrete must be able to resist cycles of freezing and thawing. In terms of mix design, difficulties are likely to arise in achieving a stable

TABLE 19.5 Recommended Maximum Water/Cementitious Material Ratios for High-Strength Concrete Containing Portland Cement and Fly Ash made with a Superplasticizer[a]

		w/cm			
		Maximum-size coarse aggregate, mm (in.)			
Average Required Strength f'_{cr}, MPa (lb/in.²)		9.5 ($\frac{3}{8}$)	12.5 ($\frac{1}{2}$)	19 ($\frac{3}{4}$)	25 (1)
48 (7000)	28-day	0.50	0.48	0.45	0.43
	56-day	0.55	0.52	0.48	0.46
55 (8000)	28-day	0.44	0.42	0.40	0.38
	56-day	0.48	0.45	0.42	0.40
62 (9000)	28-day	0.38	0.36	0.35	0.34
	56-day	0.42	0.39	0.37	0.36
69 (10,000)	28-day	0.33	0.32	0.31	0.30
	56-day	0.37	0.35	0.33	0.32
76 (11,000)	28-day	0.30	0.29	0.27	0.27
	56-day	0.33	0.31	0.29	0.29
83 (12,000)	28-day	0.27	0.26	0.25	0.25
	56-day	0.30	0.28	0.27	0.26

[a] Adapted from ACI 211.4R. Reproduced with permission.

air-void system in a mixture containing both a high cement content and a superplasticizer. The mix design process will include evaluating the compatibility between the air-entraining agent and the superplasticizer in terms of both air-void spacing and total air content. Quantities of air-entraining agent may have to be considerably higher than used for conventional concretes. However, because of the lower *w/cm* ratio, high-strength concretes contain lower amounts of freezable water than do normal-strength concretes, and as a result, lower total air contents may prove satisfactory for providing frost resistance. The principal drawback of using entrained air in high-strength concrete is its negative effect upon strength—approximately a 5% loss for each 1% increase in air content. As a general rule, the greater the cement content, the more difficult it is to recover the lost strength by reducing the *w/cm* ratio. At the high cement contents used for high-strength concretes, a significant loss in strength with the addition of entrained air is inevitable.

Proportioning Methods

The information presented earlier in this section, including Tables 19.2, 19.4, and 19.5, provides some guidance in developing initial mixture proportions. ACI 211.4R recommends developing a matrix of trial batches that include cement only, plus cement with at least two different percent replacements of mineral admixture for each mineral admixture tested. Evaluating different cements and pozzolans is advisable since the type and source will affect the mix proportions, as well as the optimum amount of cement replaced. Initial mix proportions should be established using absolute volume calculations, as described in Chapter 10. Trial batches should be adjusted first for workability, usually best handled with additional superplasticizer, and ultimately for strength. It is

often advisable to evaluate more than one superplasticizer. Coarse aggregate content should be modified if the mix is harsh or oversanded.

It is advisable to evaluate strength at 7 and 28 days, in addition to the curing time specified for the design strength. If strength is inadequate, trial batches should again be adjusted by reducing the *w/cm* ratio, modifying the percent of pozzolan replacement, or evaluating other pozzolans or cement, and perhaps selecting another coarse aggregate size.

In general (as for normal-strength concrete), the best overall properties and optimum economy are achieved through the use of the highest coarse aggregate content using the largest maximum size coarse aggregate that can be placed and consolidated, and the minimum quantity of cementitious material that will produce the desired strength. As in all aspects of concrete construction, there is not one truly optimum mix, but a series of practical and relatively economical combinations that will provide the desired properties. Determining these mixes and selecting the mix for use on the job requires an increasingly greater effort as the desired strength increases.

Evaluation of Compressive Strength

Several aspects of concrete cylinder preparation and testing need to be handled differently or given special consideration for high-strength concrete as compared to cylinders prepared for normal-strength concrete. These include specimen size, mold material, curing, end condition, and the standard deviation used to determine the average required strength, f'_{cr}.

While 150 × 300-mm (6 × 12-in.) cylinders represent the standard for determining concrete compressive strength, cylinders of this size made with high-strength concrete may possess strengths in excess of the capacity of standard laboratory compression test equipment. To overcome this drawback, high-strength concrete mixtures are often evaluated using 100 × 200-mm (4 × 8-in.) cylinders, which also meet the requirements of ASTM C 39, but which usually provide somewhat higher values of peak compressive stress than obtained with 150 × 300-mm (6 × 12-in.) cylinders. Comparisons of cylinder strengths for the same materials indicate that the smaller cylinders give compressive strengths that are typically 1 to 4% higher than those obtained with the larger specimens. Thus, when evaluated using 100-mm diameter cylinders, strength should be conservatively reduced by 4% to represent an equivalent 150-mm diameter cylinder. Construction specifications should explicitly indicate the specimen size to be used to measure f'_c.

The deformation of the mold during fabrication or handling can have a relatively greater effect on strength for high-strength concrete than for normal-strength concrete. Steel molds have been shown to give somewhat higher and generally more consistent strength results than plastic or cardboard molds, and, therefore, are recommended. Although single-use plastic molds have been shown to give satisfactory results, cardboard molds are not recommended for concrete strengths above 40 MPa (6000 lb/in.²).

Achieving a smooth bearing surface for a test specimen is normally handled by capping the ends with a sulfur compound. However, as concrete strength increases, the strength of the sulfur compound plays a greater and greater role. Evaluations of testing techniques for high-strength concrete indicate that sulfur capping compounds can be

used up to concrete compressive strengths of 100 MPa (15,000 lb/in.^{2}), if cap thickness is in the range of 1.5 to 3 mm ($\frac{1}{16}$ to $\frac{1}{8}$ in.), and all surface irregularities are removed prior to capping. Other methods of providing a smooth-bearing surface for a test cylinder include forming smooth bearing surfaces, capping at an early age with neat cement paste (which is then cured with the cylinder), or grinding the bearing surfaces. For concrete strengths on the order of 90 MPa (13,000 lb/in.^{2}), grinding results in test strengths that are about 6% higher than those produced by capping with sulfur. As pointed out in Section 14.3, unbonded end caps may not be used for strengths greater than 80 MPa (12,000 lb/in.^{2}).

The properties of the testing machine tend to play a greater role for high-strength concrete than they do for normal-strength concrete. Machines with low lateral stiffness and machines with spherically seated loading platens that continue to rotate as load is applied result in significantly reduced strengths.

Variability of Compressive Strength As discussed in Chapter 15 and earlier in this chapter, procedures for determining the average required strength of concrete for a job depend on the specified compressive strength and the standard deviation of the field concrete. For normal-strength concretes, it is often assumed that the standard deviation of the concrete compressive strength is essentially independent of the value of the compressive strength itself. For high-strength concretes, however, such an assumption tends to underestimate the standard deviation, which has been shown to depend on both the compressive strength and the age of the concrete. When good quality control procedures are used, the standard deviation generally increases as concrete strength increases. However, the increase in standard deviation is not proportional to the increase in strength, with the result that the coefficient of variation generally drops as strength increases. For ready-mix concrete operations that have experience in the production of high-strength concrete, standard deviations for 100-MPa (15,000-lb/in.^{2}) concrete will be 10 to 20% higher than for 35 MPa (5000 lb/in.^{2}) concrete.

As discussed in Chapter 15, two criteria are used by the ACI Building Code (ACI 318) to establish f'_{cr}. For $f'_c \leq 35$ MPa (5000 lb/in.^{2}), one of the criteria is that the probability be less than 1 in 100 that an individual test will be $< f'_c - 3.5$ MPa (500 lb/in.^{2}). This led to the requirement that

$$f'_{cr} = f'_c + 2.33s - 3.5 \qquad \text{MPa} \qquad (15.11a)$$

$$f'_{cr} = f'_c + 2.33s - 500 \qquad \text{lb/in.}^2 \qquad (15.11b)$$

where f'_c is the specified compressive strength and s is the standard deviation in compressive strength. The 3.5 MPa (500 lb/in.^{2}) in Eq. (15.11) is based on concretes with strengths of 20 to 35 MPa (3000 to 5000 lb/in.^{2}), for which 3.5 MPa (500 lb/in.^{2}) represents 10% or more of the compressive strength. The greater absolute gain in strength over time exhibited by high-strength concrete compared to normal-strength concrete and the accompanying increases in s requires that Eq. (15.11) be modified to

$$f'_{cr} = 0.90f'_c + 2.33s \qquad (15.12)$$

which provides a probability of 1 in 100 that an individual test will be below $0.90\,f'_c$.

The second criterion used to establish f'_{cr}, that the probability be less than 1 in 100 that the average of three consecutive tests will be below f'_c, is applicable to all concrete strengths. The corresponding requirement for f'_{cr} is

$$f'_{cr} = f'_c + 1.34s \qquad (15.10)$$

For high-strength concrete, the higher value of f'_{cr}, from Eq. (15.10) or Eq. (15.12), is used for design.

When establishing the required average strength based on laboratory batches, account must be taken for the fact that the strength of concrete produced in the field is lower than that produced in the lab. As illustrated in Figure 19.6, the strength of high-strength concrete cast in the field averages about 90% of the strength obtained in the laboratory. The ACI Building Code (ACI 318) requires that, when average required strengths are established based on laboratory batches, f'_{cr} must exceed f'_c by 7 MPa (1000 lb/in.^{2}) for $f'_c < 20$ MPa (3000 lb/in.^{2}), by 10 MPa (1200 lb/in.^{2}) for f'_c between 20 and 35 MPa (3000 and 5000 lb/in.^{2}), and by $0.1f'_c + 5$ MPa (700 lb/in.^{2}) for $f'_c > 35$ MPa (5000 *lb/in.*^{2}) (see Table 15.3, Section 15.1).

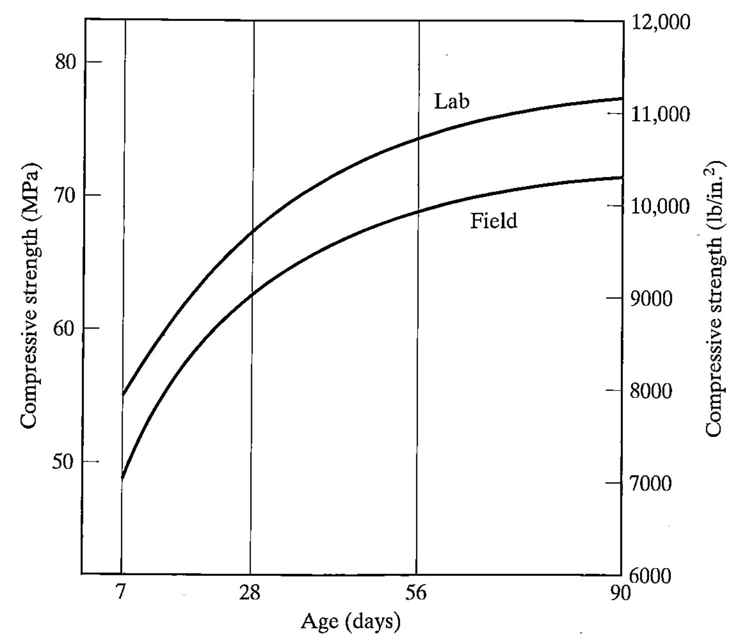


FIGURE 19.6

Comparison of laboratory concrete strengths and ready-mixed concrete strengths for 62 MPa (9000 lb/in.^{2}) concrete. [From R. L. Blick, C. F. Petersen, and M. E. Winter, in *Proportioning Concrete Mixes*, SP-46, American Concrete Institute, Detroit, MI, p. 163 (1974).]

Following the same philosophy as used to develop Eqs. (15.11) and (15.12) for establishing average required strength, ACI Committee 318 uses different lower limits for accepting concrete with strengths below f_c'. For $f_c' \leq 3.5$ MPa (5000 lb/in.^{2}), the strength of an individual test must not be less than $f_c' - 3.5$ MPa ($f_c' - 500$ lb/in.^{2}). For $f_c' > 35$ MPa (5000 lb/in.^{2}), the lower limit is 0.9 f_c'. The requirement that the average strength for all sets of three consecutive tests equals or exceeds f_c' applies for all values of f_c'.

19.2 CONSTRUCTION

High-quality construction with high-strength concrete involves the same procedures used for high-quality construction with normal-strength concrete (described in Chapter 11). However, because of the higher cementitious-material contents and the use of superplasticizers, certain aspects of the construction process should be emphasized to provide for thorough mixing and adequate consolidation, finishing, and curing.

Batching and Mixing

Because of the sensitivity of high-strength concrete to the materials used, it is especially important to have an adequate supply of all materials on hand, with minimum variations in uniformity. Cement uniformity should be evaluated in accordance with ASTM C 917, and mineral admixtures should meet their respective ASTM specifications. Variations in fine-aggregate gradation should be limited to changes in fineness modulus of ±0.2, as applies to normal-strength concrete.

Prior to batching, it is best to saturate all aggregate and then account for the free surface moisture in the batch weights. Saturated aggregate prevents additional slump loss during mixing and placing operations, and water in aggregate pores can serve as a reservoir for water of hydration after the concrete has set. Prevention of slump loss due to continued absorption of mix water by aggregates is especially important for high-strength concrete, since superplasticizers maintain slump for only a short period of time. Provision may be needed for dispensing a superplasticizer at the job site.

The batching and mixing sequence should be planned to ensure thorough dispersion and mixing of all components. High slump is often used, through the addition of superplasticizers, to aid in the mixing process. Most admixtures are more effective when added after the cement has been wetted. However, some plasticizing admixture is usually needed to obtain initial mixing of water and cement. Without an initial dose, the cement with the low water content may ball up, lowering the efficiency of the mixing operation. Satisfactory results have been obtained using traditional ribbon-feeding techniques, as has thorough mixing of the mortar constituent followed by addition of the coarse aggregate. A general rule for mixing time is to double that used for the same quantity of normal-strength concrete.

Temperature control is more important for high-strength concrete than it is for normal-strength concrete, with placement recommended at concrete temperatures of 15 to 25°C (60 to 75°F). Placement at a higher temperature risks a significant reduction in the setting time due to the heat generated by the high cement content and the progressively limited working life of superplasticizers as temperature rises. Below

10°C (50°F), naphthalene superplasticizers become viscous, causing problems with their dispersion in water, and excessive retardation may occur. If necessary, during placement, slump should be maintained by redosing with a superplasticizer since such additions will restore workability and usually result in an increase in compressive strength, as shown in Table 19.3.

Placing, Finishing, and Curing

Due to the rapid loss of workability exhibited by high-strength concretes, speed is important in all aspects of the construction process. The cohesive consistency of most high-strength concretes often makes it "sticky," so that it is difficult to provide a smooth trowel finish. A special area of difficulty involves the use of high-strength concrete containing silica fume in flatwork, such as overlays for highway bridges and parking structures. The lack of bleeding means that, as the surface water evaporates, it is not replaced from below, so that these concretes are particularly susceptible to plastic shrinkage cracking. In such cases, the area of placement needs to be limited, finishing operations completed, and protective covering provided before plastic shrinkage cracking can occur. The use of fog spray and evaporation retardants are highly advantageous in reducing the rate of evaporation.

At the low *w/cm* ratios used for high-strength concrete, complete hydration cannot take place. This, however, does not mean that curing is unimportant, since problems involved with surface drying should not be added to drying that occurs due to self desiccation. Ponding provides the best option for curing, followed by covering with a wet absorbent material, such as burlap, that is in turn protected from loss of moisture by a plastic membrane. After initial curing, preferably for at least seven days, continued curing under ambient conditions provides adequate strength gain in most cases due to the ability of the low-permeability concrete to maintain initial moisture contents at depth.

19.3 PROPERTIES

High-strength concrete differs from normal-strength concrete principally in its higher density, its greater uniformity, and the lower porosity of the cement paste matrix. The unhydrated cement particles, which are stronger than the surrounding hydration products, add significantly to strength while also reducing the permeability of the paste. As strength increases, the stiffness of the paste constituent becomes progressively closer to that of the aggregate particles. When silica fume is used, the structure of the interface between cement paste and aggregate is densified, resulting in an increase in bond strength and a decrease in permeability. Ultimately, these changes affect the engineering properties of high-strength concrete.

Compressive Behavior

An increase in the compressive strength of concrete results in an increase in the steepness of both ascending and descending branches of the stress–strain curve as well as an increase in the strain corresponding to the peak stress, as shown in Figure 13.28 (Section 13.4). The increase in the steepness of the descending branch indicates that the concrete is more brittle.

Modulus of Elasticity As discussed in Section 13.2, there is a general relationship between compressive strength and modulus of elasticity. The equation used in the ACI Building Code [Eq. (13.1)] may overestimate the modulus of elasticity for many high-strength concretes (Figure 19.7):

$$E_c = 0.043\, w_c^{1.5} \sqrt{f'_c} \quad \text{MPa} \tag{13.1a}$$

or

$$E_c = 33\, w_c^{1.5} \sqrt{f'_c} \quad \text{lb/in.}^2 \tag{13.1b}$$

A somewhat more conservative equation is recommended by ACI Committee 363 for compressive strengths between 21 and 83 MPa (3000 and 12,000 lb/in.^{2}):

$$E_c = 3320 \sqrt{f'_c} + 6900 \quad \text{MPa} \tag{19.1a}$$

$$E_c = 40{,}000 \sqrt{f'_c} + 1.0 \times 10^6 \quad \text{lb/in.}^2 \tag{19.1b}$$

A lack of generality in Eq. (13.1) or Eq. (19.1), however, along with the inherent variability of concrete properties, is illustrated in Figure 19.8, which shows that, when used for concretes containing coarse aggregates with a high modulus of elasticity [61 to

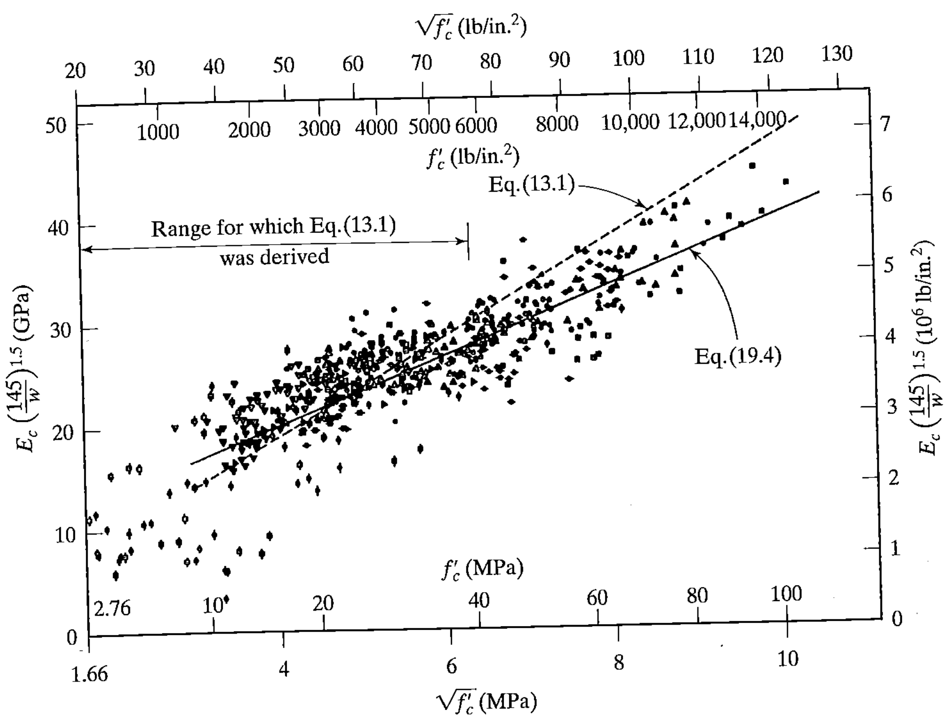


FIGURE 19.7

Modulus of elasticity versus concrete strength. [Adapted from S. Martinez, A. H. Nilson, and F. O. Slate, *Research Report*, No. 82-9, Dept. of Structural Engrg., Cornell University (1982).]

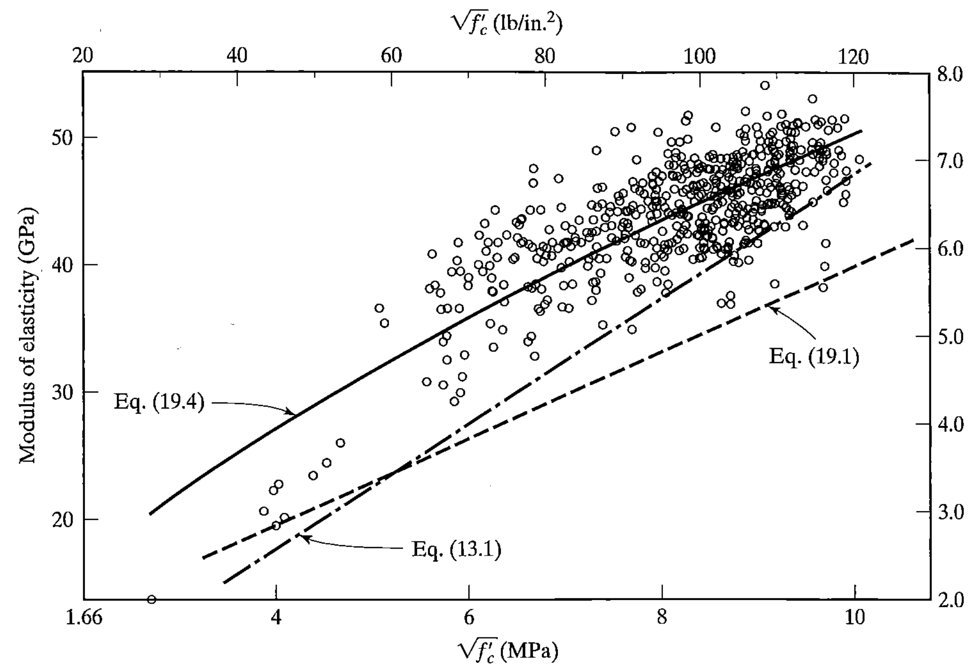


FIGURE 19.8

Modulus of elasticity versus concrete strength for normal-weight concretes. Upper curve shows the effects of coarse aggregates with high moduli of elasticity. [Adapted from J. E. Cook, *Concrete International*, Vol. 11, No. 10, pp. 67–75 (1989).]

69 MPa (8.8 to 10.0 × 10^6 lb/in.^{2})] both Eqs. (13.1) and (19.1) can greatly underestimate the modulus of elasticity of concrete. The best fit equation shown in Figure 19.8 is

$$E_c = 2.8 \times 10^{-5} w^{2.55} f_c'^{\,0.315} \quad \text{MPa} \tag{19.2a}$$

$$E_c = w^{2.55} f_c'^{\,0.315} \quad \text{lb/in.}^2 \tag{19.2b}$$

Limited comparisons indicate that Poisson's ratio is largely independent of compressive strength.

Microcracking Research on microcracking in high-strength cement pastes and concretes indicates that, at a given strain, both materials exhibit less microcracking than do the corresponding normal-strength materials. Microcracking in high-strength cement paste occurs principally through C–S–H and at the boundary between C–S–H and harder constituents. Very little cracking occurs through unhydrated cement grains. At lower magnifications, microcracking studies of high-strength concrete indicate a reduction in microcracking at the paste–aggregate boundary compared to normal-strength concrete.

Tensile and Fracture Behavior

As described in Chapter 13, both the tensile strength and fracture parameters of concrete do not increase in proportion to compressive strength. The relationships proposed by ACI Committee 363 for split-cylinder tensile strength and modulus of rupture are given in Eqs. (19.3) and (19.4), respectively, for concrete with strengths between 21 and 83 MPa (3000 to 12,000 lb/in.^{2}).

$$f'_{sp} = 0.59\sqrt{f'_c} \quad \text{MPa} \tag{19.3a}$$

$$f'_{sp} = 7.4\sqrt{f'_c} \quad \text{lb/in.}^2 \tag{19.3b}$$

$$f'_r = 0.94\sqrt{f'_c} \quad \text{MPa} \tag{19.4a}$$

$$f'_r = 11.7\sqrt{f'_c} \quad \text{lb/in.}^2 \tag{19.4b}$$

However, as described in Section 13.3, analysis of a wider range of data indicates that the best-fit expressions are

$$f'_{sp} = 0.305 f_c'^{\,0.55} \quad \text{MPa} \tag{13.5a}$$

$$f'_{sp} = 4.34 f_c'^{\,0.55} \quad \text{lb/in.}^2 \tag{13.5b}$$

$$f'_r = 0.438 f_c'^{\,2/3} \quad \text{MPa} \tag{13.6a}$$

$$f'_r = 2.30 f_c'^{\,2/3} \quad \text{lb/in.}^2 \tag{13.6b}$$

Comparisons between Eqs. (19.3) and (13.5) for splitting tensile strength and between Eqs. (19.4) and (13.6) for modulus of rupture are given in Figures 13.9 and 13.10 (Section 13.3), respectively.

Fracture Parameters Fracture parameters such as the critical stress intensity factor, K_{Ic}, and the fracture energy, G_f, increase at a significantly lower rate than does the tensile strength of concrete [Eqs. (13.5), (13.6), (19.3), (19.4)]. In one study, the compressive strength of concrete increased over 160%, from 33 to 86 MPa (4800 to 12,500 lb/in.^{2}), while K_{Ic} increased only 25% and G_f increased only 12%. Increases of 61% would be expected if these parameters increased in proportion to $\sqrt{f'_c}$. In a number of other studies, G_f remained essentially unchanged for compressive strengths between 20 and 99 MPa (2900 and 14,300 lb/in.^{2}), independent of age or *w/cm* ratio, as well as compressive strength at ages of 7 to 180 days. Research on normal-strength concrete has demonstrated that fracture parameters are highly sensitive to the maximum size of the coarse aggregate, with a reduction in coarse aggregate size from 25 to 5 mm (1 to 0.2 in.) resulting in a reduction in fracture energy of 49%, even as compressive strength increased from 33 to 40 MPa (4800 to 5800 lb/in.^{2}). Research on high-strength concrete indicates that both tensile strength and fracture energy increase with the volume of coarse aggregate, if the aggregate is of high strength. It is, however, the strength of the coarse aggregate itself that has the greatest effect on fracture energy, with increases of 100 to 300% observed as higher quality aggregates are substituted for lower quality aggregates. This fact and the general insensitivity of fracture energy to compressive strength is illustrated in Figure 19.9.

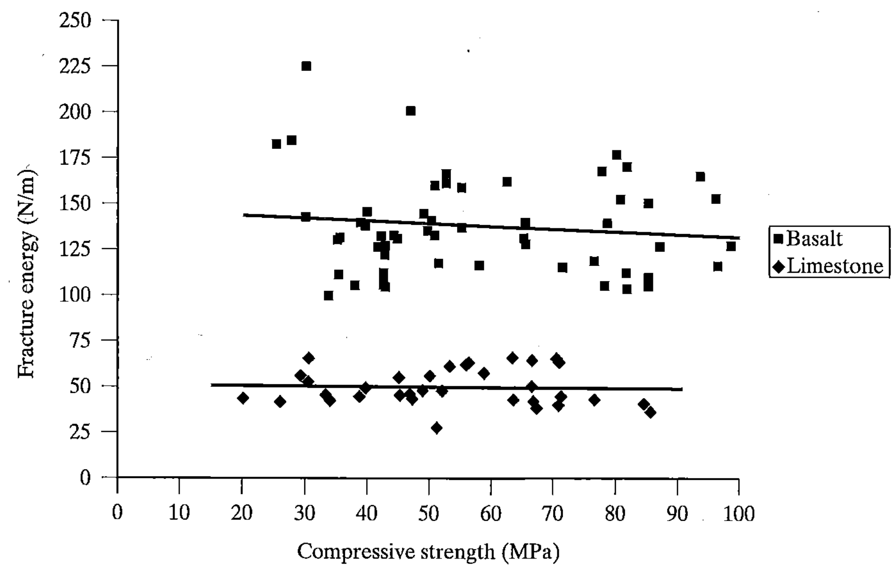


FIGURE 19.9

Fracture energy versus compressive strength. [From D. Darwin, S. Barham, R. Kozul, and S. Luan, "Fracture Energy of High-Strength Concrete," *ACI Materials Journal*, Vol. 98, No. 5, pp. 410–417 (2001).]

Shrinkage and Creep

Shrinkage The available data on shrinkage are mixed, with some indicating that high-strength concrete shrinks more than normal-strength concrete and some indicating that it shrinks less. The reason for some of the confusion can be explained by Figure 19.10, which shows shrinkage strain as a function of *w/cm* ratio for total aggregate contents of 60, 70, and 80% of the volume of concrete. It can be seen that an increase in the *w/cm* ratio results in an increase in shrinkage and that the effect becomes progressively greater as the aggregate volume decreases. Overall, however, Figure 19.10 shows that it is the total *nonaggregate* portion of concrete that controls shrinkage. Studies that involved the use of high cement contents or a decrease in coarse aggregate content in conjunction with a reduction in the *w/cm* ratio, procedures often used in early high-strength concrete research, would be expected to indicate increased shrinkage as a result of the increase in the nonaggregate portion of the concrete. On the other side, superplasticizers have been shown to reduce drying shrinkage, as has the use of longer curing times. Concrete with silica fume may exhibit higher shrinkage than concrete without silica fume. Overall, high-strength and normal concretes exhibit similar shrinkage strains, although one might expect that, in the long term, high-strength concrete would exhibit successively lower strains due to the lower total water content and lower permeability of the material.

Creep High-strength concrete differs from normal-strength concrete in the relatively lower volume of hydration products and the lower volume of free water. The process of

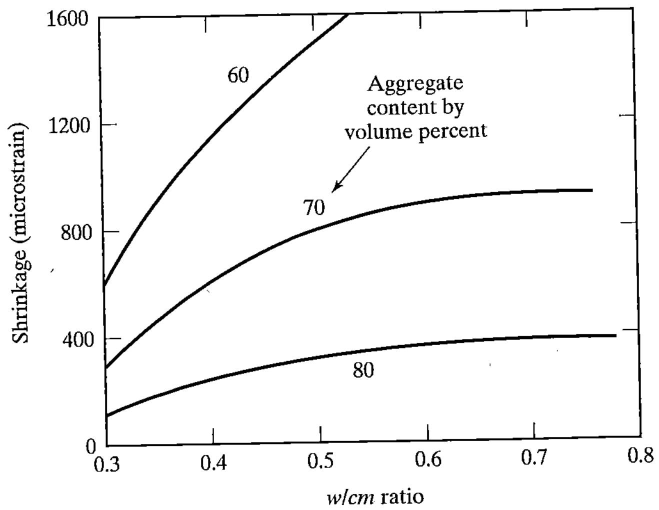


FIGURE 19.10

Effect of *w/c* ratio and aggregate content on shrinkage. [From S. T. A. Ödman, *RILEM/CEMBUREAU Intl. Colloquium on the Shrinkage of Hydraulic Concretes*, Madrid, Vol. 1 (1968).]

self desiccation tends to reduce the relative humidity of high-strength concrete below that of normal-strength concrete.

As a result of differences in microstructure and water content, high-strength concrete creeps significantly less than normal-strength concrete, when expressed in terms of both the specific creep (creep strain per unit stress) and the creep coefficient (ratio of creep strain to initial strain). This lower creep is illustrated in Figures 19.11 and 19.12. Figure 19.11 shows specific creep as a function of compressive strength for concretes with strengths between 30 and 70 MPa (4500 and 10,000 lb/in.^{2}). The ratio of specific creep for high-strength concrete to specific creep for normal-strength concrete is in the range of 0.2 to 0.35. Figure 19.12 shows the creep coefficient for concrete strengths ranging from 20.9 to 65.4 MPa (3030 to 9480 lb/in.^{2}) as a function of the stress–strength ratio. As concrete strength increases, the creep coefficient decreases and the range over which the creep coefficient is constant (i.e., independent of the stress–strength ratio) increases. As shown in the figure, concrete with a strength between 55 to 65 MPa (8000 to 9500 lb/in.^{2}) exhibits only about one-third the creep coefficient of concrete with a strength between 20 to 25 MPa (3000 to 3600 lb/in.^{2}). In building construction, high-strength concrete columns typically have about one-third of the area of the normal-strength concrete columns they replace. Thus, the total creep strain will be similar or somewhat less for columns constructed with high-strength concrete than for columns supporting the same loads, but constructed with normal-strength concrete.

Durability

Under nearly all conditions, high-strength concrete exhibits superior durability to normal-strength concrete, due principally to the much lower permeability obtained with low *w/cm* ratios and (when used) silica fume as a partial replacement for cement. The cement paste constituent of high-strength concrete exhibits greater uniformity and

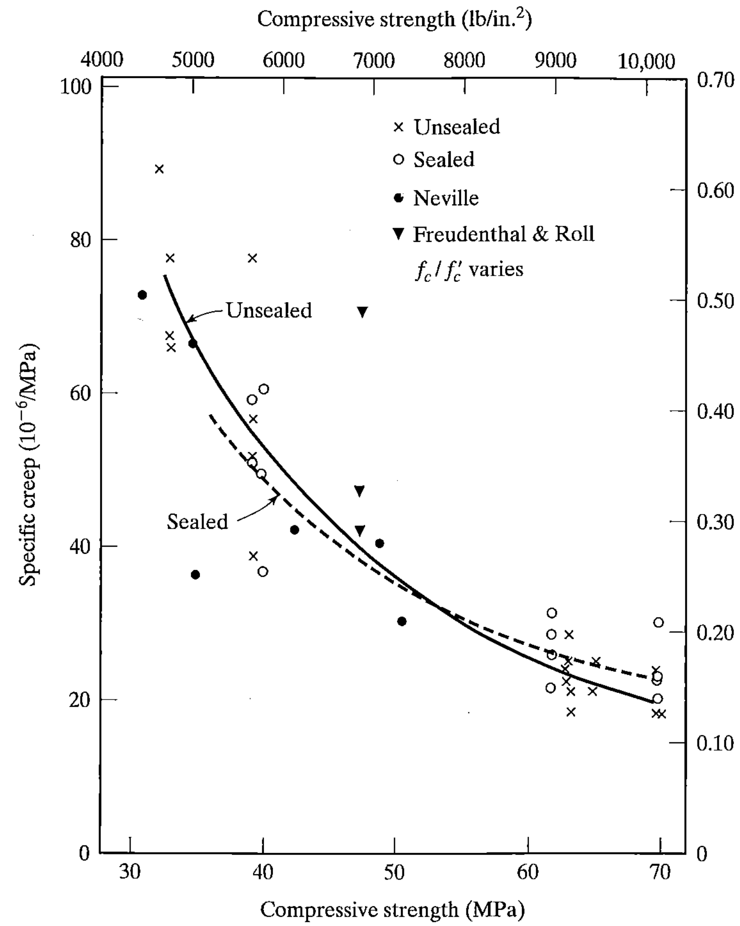


FIGURE 19.11

Specific creep versus compressive strength for concrete under load for 60 days. [Adapted from A. S. Ngab, A. H. Nilson, and F. O. Slate, *ACI Journal*, Vol. 78, No. 4, pp. 255–261 (1981).]

homogeneity, along with lower porosity, than does the paste constituent of normal-strength concrete. When pozzolans are used, calcium hydroxide (a soluble hydration product) is converted to C–S–H, and as the calcium–silica ratio is decreased, the other alkalis and aluminum are incorporated in the cement paste matrix. The denser matrix contains less freezable water, makes it more difficult for ions to penetrate the concrete, and if an adequate amount of pozzolan is used, improves the ability of the concrete to withstand the alkali–aggregate reaction.

Silica fume changes the structure of the transition zone between cement paste and aggregate, converting a region of higher porosity and increased CH content to a dense matrix of C–S–H. The denser transition zone results in a further decrease in permeability.

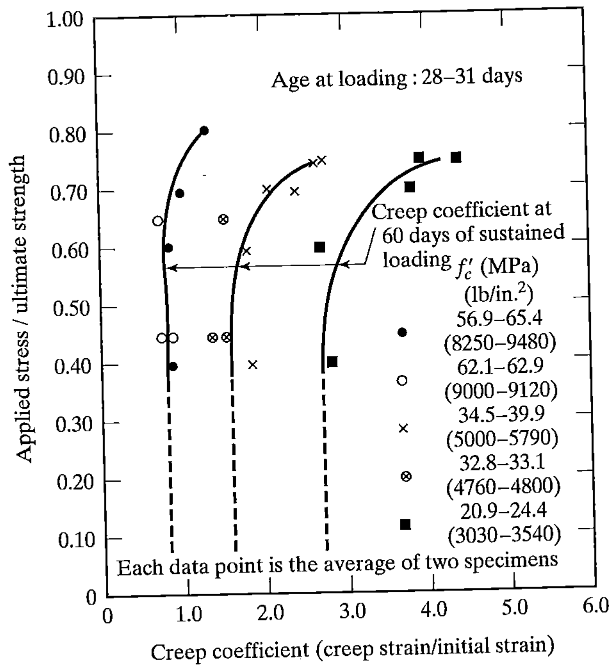


FIGURE 19.12

Stress–strength ratio versus creep coefficient for high, medium, and low-strength concretes in compression. [From M. M. Smadi, F. O. Slate, and A. H. Nilson, *ACI Materials Journal*, Vol. 84, No. 3, pp. 224–234 (1987).]

Freeze–Thaw Resistance As a general rule, a decrease in the *w/cm* ratio improves the ability of concrete to withstand cycles of freezing and thawing. Theoretically, if the *w/cm* ratio is low enough, there will not be enough freezable water in the concrete to cause damage. Studies evaluating the freeze–thaw resistance of high-strength concretes indicate that, at a *w/cm* ratio = 0.24 or less, entrained air is not required. However, for higher *w/cm* ratios, entrained air is required to provide freeze–thaw resistance. Figure 19.13 shows the change in the dynamic modulus of elasticity of non-air-entrained concrete with increasing freeze–thaw cycles using ASTM C 666 Procedure A (discussed in Section 18.4). Of the four concretes evaluated (*w/cm* ratio = 0.24, 0.27, 0.30, and 0.33), only the mixture with *w/c* ratio of 0.24 was able to withstand greater than 150 freeze–thaw cycles without significant damage. In this case, the low-*w/c* ratio concrete was able to withstand 1000 cycles with less than a 20% drop in the dynamic modulus. Figure 19.14 shows a similar comparison for concretes with a *w/cm* ratio of 0.24 containing 0, 5, and 10% silica fume. After 1000 cycles, the concretes containing 0 and 10% silica fume exhibited less than a 5% drop in the dynamic modulus, while the concrete containing 5% silica fume exhibited a 19% drop in dynamic modulus. In all cases, the performance is quite satisfactory.

Other studies have shown that adequate curing is very important to the ability of non-air-entrained high-strength concrete to withstand cycles of freezing and thawing. Research has also demonstrated that non-air-entrained concrete with a *w/cm* ratio as high as 0.27 performs quite well when tested for salt scaling under ASTM C 672. However, the overall consensus is that a water–cement ratio of 0.24 or less is required for non-air-entrained concrete to withstand cycles of freezing and thawing.

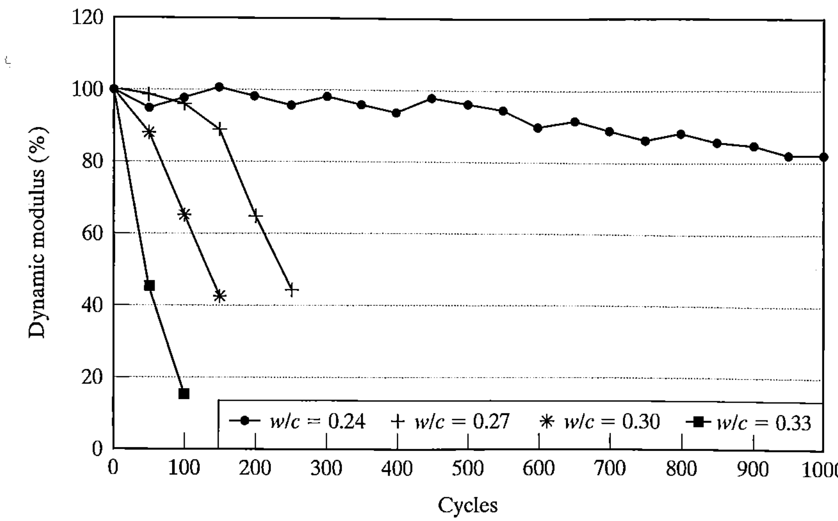


FIGURE 19.13

Freeze–thaw durability of non-air-entrained concrete without silica fume cured for 14 days prior to testing. (From Y. Li, B. W. Langan, and M. A. Ward, in *High-Performance Concrete*, ed. V. M. Malhotra, SP-149, American Concrete Institute, Detroit, MI, pp. 545–560 (1994).]

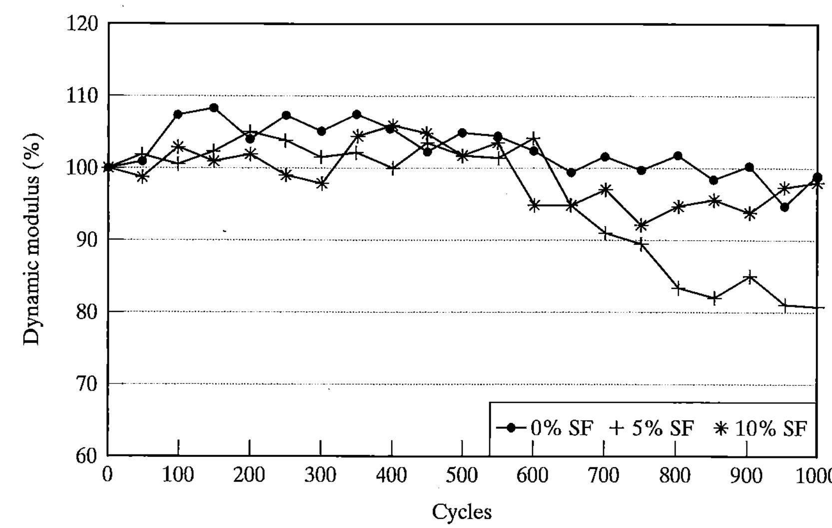


FIGURE 19.14

Freeze–thaw durability of non-air-entrained concrete with varying silica fume contents, *w/cm* ratio = 0.24, cured for 28 days prior to testing. [From Y. Li, B. W. Langan, and M. A. Ward, in *High-Performance Concrete*, ed. V. M. Malhotra, SP-149, American Concrete Institute, Detroit, MI, pp. 545–560 (1994).]

Chemical Attack Resistance to chemical attack is greatly improved as the *w/cm* ratio is decreased, principally because of the decrease in permeability. The use of pozzolans reduces the proportion of soluble CH, and silica fume has been shown to aid in withstanding attack by sulfates and weak acids. When used in adequate quantities, all pozzolans will help reduce the alkali–aggregate reaction.

Abrasion The ability of concrete to withstand surface abrasion and water erosion is improved as the strength of the concrete increases. Concrete containing silica fume has been used to improve the abrasion resistance of concrete subjected to studded tire wear and in storage facilities affected by the movement of granular materials. The U.S. Army Corps of Engineers has used concrete containing silica fume to line stilling basins subjected to heavy hydraulic abrasion and cavitation wear. Silica fume has also been used to improve the ability of concrete to withstand ice abrasion. As with normal-strength concrete, the abrasion resistance of high-strength concrete depends not only on the cement paste constituent, but on the use of hard abrasion-resistant aggregates.

Corrosion As described in Chapter 18, reinforcing steel in concrete is protected from corrosion due to the high pH of the concrete environment and the limitation of access of oxygen and moisture to the steel. The reduction in permeability provided by low *w/cm* ratios enhances this protection not only by limiting access by oxygen and moisture, but also by reducing the rate of carbonation and limiting access by chlorides. One potentially negative impact of using pozzolans to obtain high strengths is the reduction in the pH of the pore solution, with the potential loss of passivity of the reinforcing steel. It has been demonstrated, however, that this reduction in pH is more than compensated for by the decrease in permeability and increase in resistivity of the concrete obtained with the denser cement-paste matrix. As described earlier, when constructing with silica fume, special attention is required to prevent plastic shrinkage cracking from lowering the effective cover and providing a route for chlorides to reach reinforcing steel.

Fire There is some indication that high-strength concrete has a greater susceptibility than normal-strength concrete to spalling when subjected to high temperatures, such as occur when concrete is exposed to fire. This greater tendency may be due to the lower permeability of the concrete and the inability of noncombined water to expand easily. Concretes with lower permeabilities also tend to dry out more slowly and, therefore, may have a higher proportion of their noncombined water closer to the surface compared to normal-strength concretes. These points are supported by observations that show that increases in silica fume content, with the resulting decrease in permeability, tend to result in greater fire damage.

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PROBLEMS

19.1. Discuss the types of members in which high-strength concrete can be used effectively and those in which it cannot.

19.2. Discuss the engineering advantages and disadvantages of using high-strength concrete in place of normal-strength concrete.

19.3. Why does high-strength concrete gain a greater percentage of its long-term strength more rapidly than normal-strength concrete?

19.4. What will the average required strength, f'_{cr} be for concrete with a specified compressive strength of 90 MPa (13,000 lb/in.^{2}) and standard deviation of 4.8 MPa (700 lb/in.^{2}) according to the ACI Building Code?

19.5. Will all Type I cements perform in essentially the same manner in the production of high-strength concrete? Discuss.

19.6. Why is it advisable to use lower water contents in the production of high-strength concrete than used in the production of normal-strength concrete?

19.7. What properties of fine aggregate are desirable for the production of high-strength concrete?

19.8. How does the size and quantity of coarse aggregate in a high-strength concrete mix compare to that in normal-strength concrete mixes?

19.9. What properties of silica fume make it the mineral admixture of choice for concretes with compressive strengths of 90 MPa (13,000 lb/in.^{2}) and higher?

19.10. Discuss the uses of chemical admixtures in high-strength concrete.

19.11. In what ways does plastic high-strength concrete differ in behavior from plastic normal-strength concrete?

19.12. Discuss the need to provide entrained air in high-strength concretes that are exposed to cycles of freezing and thawing.

19.13. Can the standard procedures used to test concrete in compression be used without modification when evaluating high-strength concrete? Explain.

19.14. What changes or areas of emphasis are required in the construction process when using high-strength concrete?

19.15. Other than absolute differences in strength, how does high-strength concrete differ from normal-strength concrete in compression? In tension?

19.16. Compare high-strength concretes with normal-strength concretes in terms of shrinkage and creep.

19.17. Does high-strength concrete have any advantages over normal-strength concrete in terms of durability? Explain.

CHAPTER 20

Concretes for Special Applications

20.1 LIGHTWEIGHT CONCRETES

Lightweight concretes can be divided into structural lightweight concretes and ultra-lightweight concretes used for nonstructural purposes. ACI Committee 213 makes three divisions (Figure 20.1) on the basis of strength and unit weight: low-density, low-strength concrete used for insulation; moderate-strength lightweight concrete used for concrete block and other applications where some useful strength is desirable; and structural lightweight concrete.

The various types of lightweight aggregates were discussed in Chapter 7 and are summarized in Figure 20.1. However, lightweight concretes can also be obtained by other means. Low densities are achieved by a high porosity within the concrete, and this porosity need not be confined to the aggregate. Aerated (foamed or cellular) concrete has a uniform distribution of air voids throughout the paste or mortar, while "no-fines" concrete or lightly compacted concretes also contain large, irregular voids. Aerated concrete is reviewed in this section; no-fines concrete is discussed later in the chapter as a special kind of gap-graded concrete. Table 20.1 summarizes the various kinds of lightweight concretes.

The use of lightweight concretes in the United States dates back to the early 1900s. Expanded clays and shales were developed commercially by S. H. Hayde (the Haydite process) and were used for shipbuilding during World War I. The Park Plaza Hotel in St. Louis was an early example of lightweight concrete construction in the 1920s. Clinker aggregate was also developed prior to World War I and used mostly in lightweight block, and foamed slag was produced commercially in the late 1920s. Since the 1950s, lightweight concrete has been used regularly in multistory buildings and other large structures. Some of the more notable examples are the Bank of America Corporate Center, Charlotte (Figure 20.2a); the Watergate Apartments, Washington, D.C.; the Lake Point Towers, Chicago; and the Assembly Hall, University of Illinois at Urbana–Champaign (Figure 20.2b). Lightweight concrete is also used for applications as diverse as highway bridges and offshore drilling platforms.

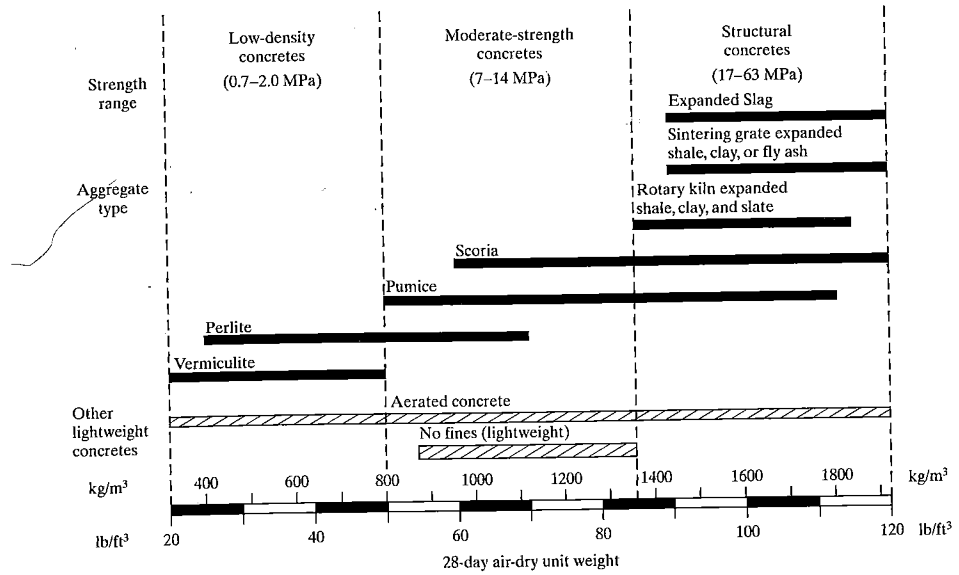


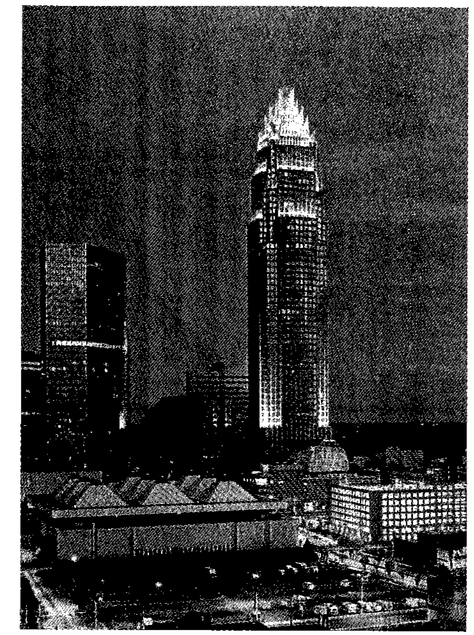
FIGURE 20.1

Classification of lightweight concretes (MPa × 145 = lb/in.^{2}). [Adapted from ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete (ACI 213R-87)," *ACI Manual of Concrete Practice*, Part 1, American Concrete Institute, Farmington Hills, MI (2001).]

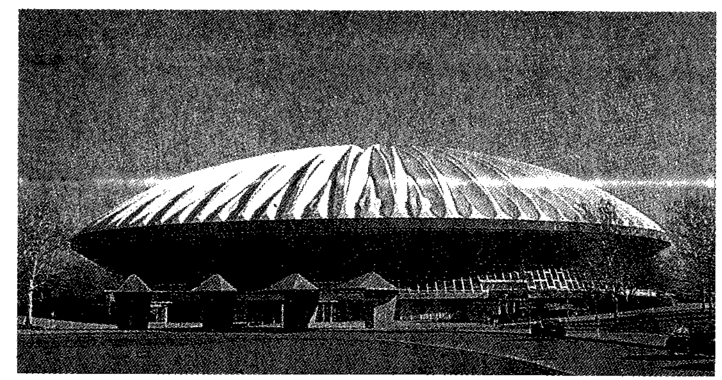
TABLE 20.1 Properties of Different Types of Lightweight Concrete

Type of Lightweight Concrete	*Type of Aggregate*	*Dry-Rodded Unit Weight of Aggregate (kg/m³)*[a]	*Compressive Strength at 28 Days (MPa)*[a]	*Unit Weight of Concrete (kg/m³)*[a]	*Thermal Conductivity (W/m · K)*[a]
Aerated concrete	–	–	0.5–24	300–1900	0.05–1.30
Partially compacted	Expanded vermiculite and perlite	65–250	0.4–3.0	300–1120	0.07–0.10
	Pumice	500–900	1.0–3.0	800–1800	0.15–0.30
	Expanded slag	500–1000	1.0–4.5	960–1520	0.16–0.42
	Sintered pulverized-fly ash	600–1000	2.0–5.5	1120–1280	0.17–0.30
	Expanded shale, clay, slate	550–1050	4.5–6.6	960–1200	0.27–0.42
	Clinker	720–1040	1.7–5.5	720–1520	0.22–0.42
No-fines concrete	Natural aggregate	1350–1600	3.0–14	1600–2000	–
	Lightweight aggregate	500–1050	2.0–5.5	880–1360	–
Structural lightweight aggregate concrete	Expanded slag	500–1000	8.0–50	1440–2080[b]	0.34–0.74
	Sintered pulverized-fly ash	600–1000	11–63	1440–1900[b]	0.52–1.10
	Expanded shale, clay, slate	550–1050	11–50	1360–1840[b]	0.51–0.95

[a](kg/m^3) × 0.062 = lb/ft^3; MPa × 145 = lb/in.^{2}; W/m · K × 0.58 = Btu/ft · h · °F.
[b]The denser concretes are obtained by replacing some of the lightweight fine aggregate with natural sand.



(a)



(b)

FIGURE 20.2

(a) Bank of America Corporate Center, Charlotte, North Carolina. (Designed by Walter P. Moore and Associates. Photograph by Timothy Hursley, Little Rock, Arkansas.) (b) Assembly Hall, University of Illinois at Urbana–Champaign. (Photograph by David A. Lange.)

Structural Lightweight Concrete

Structural lightweight concretes are made with both coarse and fine lightweight aggregates, but it is common with the higher-strength concretes to replace all or part of the fine fraction with normal-weight sand. Such a replacement will increase the unit weight of the concrete by as much as 320 kg/m^3 (20 lb/ft^3). Although synthetic lightweight aggregates are generally more expensive than normal-weight aggregates, the increased strength-to-weight ratio offers sufficient overall saving in materials, through the reduction

of dead load to more than offset the higher aggregate cost per cubic meter of the concrete. Lower total loads mean reduced supporting sections and foundations, and less reinforcement.

Engineering Properties The engineering properties of lightweight concretes depend to a large extent on the materials used in mix design. As seen in Figure 20.1, with some lightweight aggregates, there is no difficulty in obtaining concrete strengths of 50–63 MPa (7000–9000 lb/in.^{2}), in spite of the high porosity and inherent weakness of the aggregate. A relationship between strength and density exists for lightweight concrete, but depends on the particular aggregate used (see Figure 20.3) and the amount of normal-weight sand. Low *w/cm* ratios are required to achieve the higher strengths, but the high absorption of most lightweight aggregates makes it difficult to exactly calculate the *w/cm* ratio of the paste. The need for a lower *w/cm* ratio in the paste to obtain high strengths means that generally higher cement and mineral admixture contents are needed for structural lightweight concretes compared to normal-weight concretes of the same strength (see Table 20.2). In addition, the physical characteristics of lightweight aggregates are such that more paste is often required to provide good workability.

The fracture of lightweight concrete is somewhat different from that of ordinary concrete. Failure commonly occurs through, rather than around, the aggregate. The

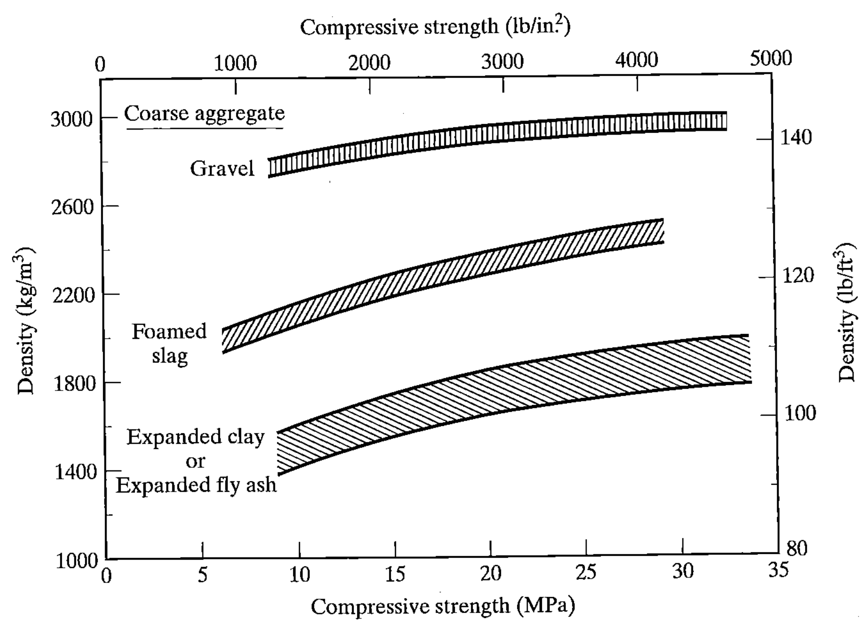


FIGURE 20.3

Relationship between density and compressive strength (150-mm test cubes). [Adapted from A. Short and W. Kinniburgh, *Lightweight Concrete*, 3d ed., Applied Science Publishers, Ltd., London (1978).]

TABLE 20.2 Cementitious Material Contents of Lightweight and Normal-Weight Concretes

Compressive Strength MPa	(lb/in.^{2})	Cementitious Material Content kg/m^3 (lb/yd)^3 Lightweight		Normal Weight	
17	(2500)	210–310	(350–520)	210–250	(350–420)
21	(3000)	240–340	(400–570)	210–300	(350–500)
28	(4000)	300–400	(500–670)	240–340	(400–570)
35	(5000)	360–450	(600–760)	300–400	(500–670)
42	(6000)	410–510	(690–860)	350–460	(590–770)

strengths of the aggregate and the cement paste are more nearly equal, and in the stronger concretes, the paste strength may exceed the aggregate strength. It has been found that the strength of lightweight concrete depends on the volume fraction of the lightweight aggregate. For concrete made with normal-weight sand, the following empirical relationship has been found:

$$\sigma_c = \sigma_a^n \cdot \sigma_m^{(1-n)} \tag{20.1}$$

In this equation σ_c, σ_a, and σ_m are the compressive strengths of the concrete, aggregate, and mortar, respectively, and n is the volume fraction of the lightweight coarse aggregate.

Lightweight aggregates have low moduli of elasticity because of their high porosity. Consequently, the elastic modulus of lightweight concrete will be lower than that of normal-weight concrete. Generally, values are in the range 10 to 17 GPa (1.5 to 2.5 × 10^6 lb/in.^{2}), about one-third to two-thirds those of normal-weight concrete. The exact value depends on the nature of the aggregates used, and thus the range of variation is somewhat greater for a given ultimate compressive strength. The equation used in the ACI Building Code to estimate the elastic modulus [Equation (13.1), Section 13.2] may not approach the actual value more closely than ±20%. The lower elastic modulus of lightweight aggregates would also offer less restraint to time-dependent deformations such as drying shrinkage and creep. On the average, creep or shrinkage strains of lightweight concrete tend to be greater than for normal-weight concrete (Figure 20.4). It must be remembered, however, that there is considerable variation in creep and shrinkage among concretes of a given density, with the magnitude depending on the cement content, the *w/cm* ratio of the paste, the modulus of elasticity of the aggregate, and the rate of moisture loss.

Physical Properties and Durability The coefficient of thermal expansion of lightweight concrete is much the same as that for normal-weight concrete, but its thermal conductivity (see Table 20.1) is considerably lower because of the large amount of void space. The thermal conductivity depends on unit weight (Figure 17.4, Section 17.1). The lower thermal conductivity means that lightweight concretes are generally more fire-resistant than are normal-weight concretes.

Lightweight aggregates are more friable than most rocks, so that lightweight concretes are generally not suitable for heavy wear. However, many synthetic aggregates

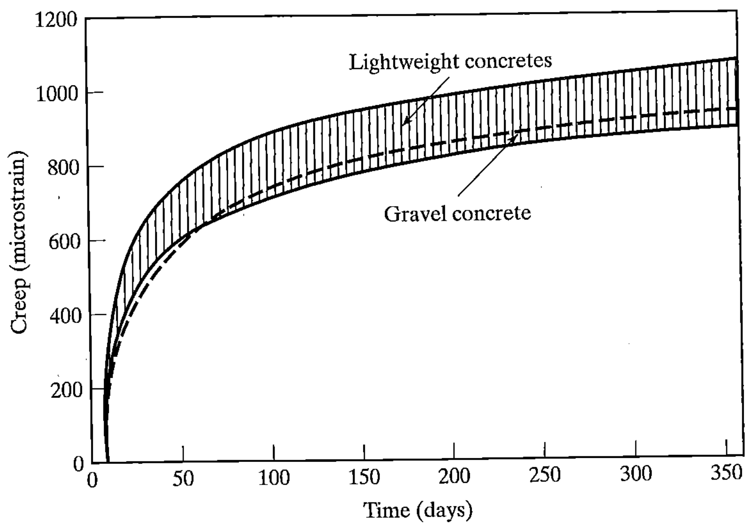


FIGURE 20.4

Comparison of creep of lightweight and normal-weight concrete. [Adapted from A. Short and W. Kinniburgh, *Lightweight Concrete*, 3d ed., Applied Science Publishers, Ltd., London (1978).]

have quite hard surfaces, and lightweight concretes can perform just as well as normal-weight concretes under less rigorous conditions of wear. The use of natural sand and the development of high compressive strengths improve the abrasion resistance.

The freeze–thaw resistance of lightweight concrete is similar to that of ordinary concrete. Air entrainment should be used whenever the concrete will be exposed to freezing and thawing. The moisture content of the aggregates can be critical, because when aggregates are close to saturation, freezing of water in the aggregate pores will force water out of the aggregate particle into the surrounding paste. The resulting hydraulic pressure may cause tensile failure if sufficient entrained air is not present to accommodate the excess water. To avoid this situation, the aggregates should have as low a moisture content as is practical during mixing, or the concrete should have ample time to dry out before being exposed to freezing temperatures. The resistance to deicer-salt scaling is also similar to normal-weight concrete. Resistance is improved by air entrainment, low *w/cm* ratios, adequate curing, and a period of drying before service. Otherwise, there are no special durability problems associated with lightweight concretes. The tendency for such concretes to have a greater absorption of water bears no relation to their durability. The higher absorption is a reflection of the porous aggregates and does not indicate a higher concrete permeability, since the latter is controlled by the porosity of the cement paste.

Fresh Concrete Properties Lightweight concrete has essentially the same properties in the plastic state as does normal-weight concrete. Mixes tend to be somewhat harsher than ordinary concrete mixes because of the nature of the synthetic aggregates. Thus, air entrainment may be desirable solely to obtain improvements in workability.

The addition of sand may be needed to improve finishability if the lightweight aggregate is deficient in fines passing the 600-μm (No. 30) sieve. Slump loss can be a severe problem when the aggregates continue to absorb large quantities of water after mixing. This condition can generally be avoided by batching the aggregate in a damp condition or by mixing the aggregate with about two-thirds of the mix water before adding the cement and the balance of the water.

Slump should be limited to a maximum of 100 mm (4 in.), because higher slumps tend to cause segregation of the lightest coarse aggregate particles rather than of the paste. It must be remembered that lightweight concretes will tend to have slightly lower slumps for a given workability because of their lower density. When placing and finishing the concrete, care should be taken to avoid segregation by using practices recommended for normal-weight concrete.

Mix Design of Structural Lightweight Concrete The particular properties of lightweight aggregates pose special problems in calculating proportions for lightweight concrete. As a result, the absolute volume method, which is the basis of the ACI method for proportioning normal-weight concrete, cannot be used with the same confidence for lightweight concrete. This is due to two major factors: (1) variations in bulk specific gravity (BSG) and (2) changes in moisture content.

The BSG of normal-weight aggregates is essentially independent of particle size. Thus, when the grading curve is determined on a weight basis (by sieve analysis), it can be converted directly to a volume basis, since it is really volume relationships that are important in proportioning. However, the constancy of the BSG does not hold with lightweight aggregates because the degree of porosity varies with particle size. The variation is particularly marked in the fine aggregate fraction (see Table 20.3) and means that the grading curve on a volume basis is different from that on a weight basis. In the example in Table 20.3, the fineness modulus as determined by weight is 3.03, whereas by volume it is 3.23. Therefore, the lightweight sand is really coarser than the value 3.03 indicates, and a greater weight of material should be retained on the finer sieve sizes to provide the same volumetric rating as a normal-weight sand.

Because of the excessive absorption of many lightweight aggregates (absorption capacity 10 to 20%), the BSG will also vary markedly with changing moisture content.

TABLE 20.3 Typical Sieve Analysis of Lightweight Fine Aggregate

Sieve Size	BSG[a]	Percent Retained By Wt.	Percent Retained By Vol.	Cumulative Percent Retained By Wt.	Cumulative Percent Retained By Vol.
4.75 mm (No. 4)	1.40	0	0	0	0
2.36 mm (No. 8)	1.55	22	26	22	26
1.18 mm (No. 16)	1.78	24	25	46	51
600 μm (No. 30)	1.90	19	19	65	70
300 μm (No. 50)	2.01	14	13	79	83
150 μm (No. 100)	2.16	12	10	91	93
Passing 150 μm (No. 100)	2.40	10	7	100	100
Fineness modulus	–	–	–	3.03	3.23

[a]SSD basis.

Furthermore, the amount of absorption that occurs during mixing is not easy to determine accurately. The amount of water absorbed by the aggregate depends on both its initial moisture content and the time of exposure to water in the fresh paste. It is thus necessary to adopt design procedures that take these points into account. ACI Standard Practice 211.2 provides a guide for estimating the proportions for the first mix and procedures for adjusting quantities for subsequent mixes. ACI 211.2 describes two methods for mix proportioning. The "mass" or "weight" method (similar to the mass method used for normal-weight concrete) is used for mixes that contain lightweight coarse aggregate and normal-weight fine aggregate. The "volumetric" method is used for mixes that contain all lightweight aggregate or a combination of lightweight and normal-weight aggregates.

First estimation of quantities using mass (weight) method. This procedure is used for so-called "sand lightweight" concrete, consisting of lightweight coarse aggregate and normal-weight fine aggregate. The procedure closely follows the ACI method of mix design described in Section 10.3, using the "mass" method to determine the quantity of fine aggregate. To follow the procedure, recommended slumps are obtained using Table 10.1, and the maximum size aggregate is selected as described in Chapter 10. Mixing water and air content are estimated using Table 20.4, which replaces Table 10.2. Water–cementitious material ratios are selected using Tables 10.3 and 10.4. The volume of dry-rodded coarse aggregate per unit volume of concrete is increased to the values shown in Table 20.5, which replaces Table 10.8 in the mix design process.

The initial fine aggregate content is based on the estimated weight of a cubic meter (cubic yard) of concrete. This depends on the relative density of the lightweight coarse aggregate, which is expressed in terms of an approximation of the bulk specific gravity known as the *specific gravity factor* (SGF). The SGF is defined as

$$\text{SGF} = \frac{\text{weight of aggregates}}{\text{effective volume of aggregates}} \qquad (20.2)$$

and is determined using procedures similar to those in ASTM C 127 and C 128 (see Section 7.1) with precautions taken to ensure that floating aggregate remains confined in the suspension cage when using ASTM C 127. The procedure allows water to be absorbed by the aggregate before the displaced volume is measured. Thus, the SGF is not a true specific gravity, since its value incorporates compensation for absorption of free water by the aggregates, but it is used in exactly the same way to calculate volume relationships. The SGF, which can be determined for lightweight fine as well as coarse aggregates, depends on the initial moisture content of the aggregate (Figure 20.5). The relationship must be determined for the aggregates being used and is typically obtained 5, 10, and 30 min after placement of the aggregate in the pycnometer or suspension basket. Values obtained after 10 min are typically used for design.

First estimates of lightweight concrete mass (weight) in kg/m^3 (lb/yd^3) are given in Table 20.6 as a function of the specific gravity factor of the coarse aggregate and the air content of the concrete. The fine aggregate content is then obtained by subtracting the weight of coarse aggregate, water, and cementitious material from the estimated weight obtained from Table 20.6.

TABLE 20.4 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Lightweight Aggregates[a, b]

Slump mm	Slump in.	9.5 mm	(3/8 in.)	12.5 mm	(1/2 in.)	19 mm	(3/4 in.)
		Water, kg/m^3 (lb/yd^3) of Concrete for Indicated Nominal Maximum Sizes of Aggregate					
Non-Air-Entrained Concrete							
30–50	1–2	210	(350)	200	(335)	185	(315)
80–100	3–4	230	(385)	215	(365)	200	(340)
130–150	5–6	235	(400)	220	(375)	210	(350)
Approximate amount of air in non-air-entrained concrete, %		3		2.5		2	
Air-Entrained Concrete							
30–50	1–2	180	(305)	175	(295)	165	(280)
80–100	3–4	200	(340)	195	(325)	180	(305)
130–150	5–6	210	(355)	200	(335)	185	(315)
Recommended average total air content, %, for level of exposure							
Mild exposure		4.5		4.0		4.0	
Moderate exposure		6.0		5.5		5.0	
Severe exposure[c]		7.5		7.0		6.0	

[a]Adapted from ACI 211.2. Reproduced with permission.
[b]These quantities of mixing water are for use in computing cement factors for trial batches. They are maxima for reasonably well-shaped, angular, coarse aggregates graded within limits of accepted specifications.
[c]These values are based on the criterion that 9% air is needed in the mortar phase of the concrete. If the mortar volume will be substantially different from that determined in this recommended practice, it may be desirable to calculate the needed air content by taking 9% of the actual mortar volume.

TABLE 20.5 Volume of Lightweight Coarse Aggregate per Unit Volume of Concrete[a, b]

Maximum Size of Aggregate mm	Maximum Size of Aggregate in.	Volume of Oven-Dry Loose Coarse Aggregate[b] per Unit Volume of Concrete for Different Fineness Moduli of Sand: 2.40	2.60	2.80	3.00
9.5	3/8	0.58	0.56	0.54	0.52
12.5	1/2	0.67	0.65	0.63	0.61
19	3/4	0.74	0.72	0.70	0.68

[a]Adapted from ACI 211.2. Reproduced with permission.
[b]Volumes are based on aggregates in oven-dry loose condition as described in ASTM C 29 for Unit Weight of Aggregate. These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For more workable concrete, such as may sometimes be required when placement is to be by pumping, they may be reduced up to 10%.

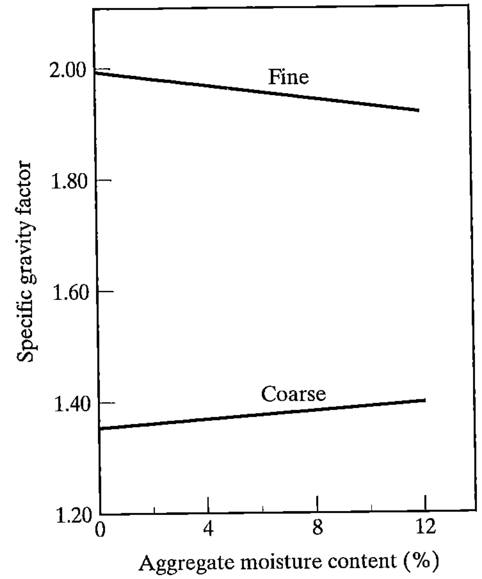


FIGURE 20.5

Variation of specific gravity factor with moisture content. [From ACI Committee 211, "Standard Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 211.2-98)," *ACI Manual of Concrete Practice*, Part 1, American Concrete Institute, Farmington Hills, MI (2001).]

TABLE 20.6 First Estimate of Weight of Fresh Lightweight Concrete Comprised of Lightweight Coarse Aggregate and Normal-Weight Fine Aggregate[a]

Specific Gravity Factor	*First Estimate of Lightweight Concrete Weight kg/m³ (lb/yd³)*[b] *Air-Entrained Concrete* 4 percent	6 percent	8 percent
1.00	1595 (2690)	1560 (2630)	1520 (2560)
1.20	1680 (2830)	1645 (2770)	1610 (2710)
1.40	1770 (2980)	1725 (2910)	1690 (2850)
1.60	1850 (3120)	1810 (3050)	1775 (2990)
1.80	1935 (3260)	1900 (3200)	1855 (3130)
2.00	2025 (3410)	1980 (3340)	1940 (3270)

[a] Adapted from ACI 211.2. Reproduced with permission.

[b] Values for concrete of medium richness [325 kg of cement per m^3 (550 lb per yd^3)] and medium slump with water requirements based on values for 80–100-mm (3–4-in.) slump in Table 20.4. If desired, the estimated weight may be refined as follows, if necessary information is available: For each 6 kg (10 lb) difference in mixing water from Table 20.4, correct the weight per m^3 (yd^3) 9 kg (15 lb) in the opposite direction; for each 60 kg (100 lb) difference in cement content from 325 kg (550 lb), correct the weight per m^3 (yd^3) 9 kg (15 lb) in the same direction.

First estimation of quantities using volumetric method. When all of the aggregate is lightweight, the recommendations must, by necessity, be somewhat more general than described, because of the wide range of properties among lightweight aggregates. The supplier of the aggregate is usually a good source of information for the particular aggregates used. Using the volumetric method, the cement content is estimated on the basis of the desired strength. The values for cementitious material content given for lightweight concrete in Table 20.2 are tabulations of values in ACI 211.2 that are based on field experience. It can be seen from the range of values that these estimates have very wide limits and, if more precise information is not available, a series of trial batches will be needed to determine the optimum cement content. Using this procedure, strength is not estimated from the *w/c* ratio because that quantity is not determined. Sufficient water is added to provide workability for proper placing, consolidating, and finishing without segregation, and to be consistent with strength requirements. Slump requirements (Table 10.1, Section 10.3) are the same as used in the mass method. No requirements are laid down for durability, but air entrainment should be used for freeze–thaw protection, with air contents as specified in Table 20.4. The total volume of aggregates required is in the range 1.04 to 1.19 m^3/m^3 (1.04 to 1.19 yd^3/yd^3), measured as the sum of the uncombined volumes of coarse and fine aggregates on a dry–loose basis. [This figure is greater than 1 m^3 (1 yd^3) because it includes the space that will be occupied by the paste, as well as some additional void space because the aggregates are not compacted.] The exact volume depends on grading, shape, texture, and surface porosity and will be least (i.e., lowest paste requirements) with well-distributed size gradation, well-rounded shape, and low surface porosity, but it also depends on the required cement, air, and water contents. The percentage of fine aggregate is generally somewhat higher than in normal-weight concrete and also depends on grading, shape, and porosity. Use of normal-weight sand with this procedure allows the proportion of fine aggregate to be decreased.

Table 20.7 demonstrates application of this procedure to determine the weights for an initial trial batch of concrete with a compressive strength of 28 MPa (4,000 lb/in.^{2}) and a slump of 100 mm (4 in.). The goal is to produce concrete with a unit weight of no more than 1680 kg/m^3 (105 lb/ft^3) on an air-dry basis. From Table 20.2, a cement content of 350 kg/m^3 (583 lb/yd^3) is estimated. The lightweight coarse and fine aggregate and normal-weight fine aggregates have damp, loose unit weights of 753, 821, and 1634 kg/m^3 (47, 51, and 102 lb/ft^3), respectively. Based on experience with these materials, the total volume of damp, loose aggregate per m^3 of concrete will be taken as 1.15 m^3, consisting of 0.62 m^3 of lightweight coarse aggregate, 0.18 m^3 of lightweight fine aggregate, and 0.35 m^3 of normal-weight fine aggregate. After mixing, the actual unit weight of the trial batch is 1717 kg/m^3 (107 lb/ft^3), producing a yield of 1.012 m^3 with an air content of 6.1%. Final adjustment of the quantities is also shown in Table 20.7.

Adjustment of mixes. Once a successful trial batch has been completed, it may be necessary to make adjustments in the field or laboratory to compensate for variations in materials (e.g., moisture content of the aggregate) or to make changes in the concrete properties. When the mass method is used for design, corrections are made

TABLE 20.7 Estimation of First Trial Batch Using the Volumetric Method

	First Trial Batch Weights, Damp, Loose, kg	*Adjusted Weights, Damp, Loose, kg/m³*
Cement	350	$\frac{350}{1.102} = 346$
Coarse lightweight aggregate	0.62 × 753 = 467	$\frac{467}{1.012} = 461$
Fine lightweight aggregate	0.18 × 821 = 148	$\frac{148}{1.012} = 146$
Fine normal-weight aggregate	0.35 × 1634 = 572	$\frac{572}{1.012} = 565$
Added water (100 mm slump)	180	$\frac{180}{1.012} = 178$
Total weight	1717	1696

much in the same way as they are for normal-weight concrete. Water should be increased or decreased by 6 kg/m³ (10 lb/yd³) for each required increase or decrease of 25 mm (1 in.) in slump. Water should be decreased or increased by 3 kg/m³ (5 lb/yd³) for each 1% increase or decrease in air content. Adjustments in fine aggregate can be based on the best estimate of the actual unit weight of the concrete or can be calculated to maintain the absolute volume of the mix based on the *changes* in the quantities of the individual constituents. Adjustments based on changes in absolute volume also work well for modifying designs based on the volumetric method.

ACI 211.2 provides additional guidance for adjusting mixes. If the changes are small, the rules of thumb given in Table 20.8 show the approximate amount of change required without affecting other properties of the concrete. These values are approximations for guidance only. When such changes are made, the aggregate content should be adjusted to ensure that yield is maintained.

An example illustrating a change in the cement content of a concrete mix is given in Table 20.9. In this example, the strength of the trial mix is too low; therefore, the cement is raised from 415 to 445 kg/m³. It can be seen in these calculations that the BSG of cement is used to calculate its effective volume in the normal way and that the SGF of the fine aggregate at the appropriate moisture content is used for the same purpose.

TABLE 20.8 Guidelines for Adjustment of Lightweight Concrete Mixes

Change Required	*Amount of Change*	*Changes in Other Quantities per m³ (yd³) of Concrete*
Proportion of fine aggregate	+1%	+2 kg (3 lb) water +1% cement
Air content	+1%	−3 kg (5 lb) water
Slump	+25 mm (1 in.)	+6 kg (10 lb) water +3% cement

TABLE 20.9 Adjustment for Change in Cement Content (SI Units)[c]

	Original Mix Dry Basis[a] $M_c = 0\%$, $S_{c0} = 1.34$	$M_f = 0\%$, $S_{f0} = 1.99$
	(1) Weight[b] *(kg)*	*(2) Effective Displaced Volume*[b] *(m³)*
Cement	415	$\frac{415}{1000 \times 3.15} = 0.132$
Air (5.5%)	—	0.055
Coarse aggregate	351	$\frac{351}{1000 \times 1.34} = 0.262$
Fine aggregate	549	$\frac{549}{1000 \times 1.49} = 0.276$
Added water	275	0.275
Total	1590	1.000

	Adjusted Mix Dry Basis $M_c = 0\%$, $S_{c0} = 1.34$	$M_f = 0\%$, $S_{f0} = 1.99$	*Adjusted Mix Damp Basis* $M_c = 1.5\%$, $S_{c1\frac{1}{2}} = 1.35$	$M_f = 4\%$, $S_{f4} = 1.97$
	(4) Weight (kg)	*(3) Effective Displaced Volume (m³)*	*(5) Weight (kg)*	*(6) Effective Displaced Volume (m³)*
Cement	445	$\frac{445}{1000 \times 3.15} = 0.141$	445	0.141
Air (5.5%)	—	0.055	—	0.055
Coarse aggregate	351	0.262	351 × 1.015 = 356	$\frac{356}{1000 \times 1.35} = 0.264$
Fine aggregate	0.267 × 1000 × 1.99 = 531	1.000 − 0.733 = 0.267	531 × 1.04 = 552	$\frac{552}{1000 \times 1.97} = 0.280$
Added water	275	0.275	1000 × 0.260 = 260	1.000 − 0.740 = 0.260
Total	1602	1.000	1613	1.000

[a] M_c, M_f = moisture contents of coarse and fine aggregate; S_{c0}, $S_{c1\frac{1}{2}}$ = specific gravity factor for coarse aggregate, dry, or with a moisture content of 1.5%, respectively; S_{f0}, S_{f4} = specific gravity factor for fine aggregate, dry, or with a moisture content of 4%, respectively.

[b] kg × 2.2 = lb; m³ × 35.3 = ft³.

[c] The sequence of calculations is determined by observing the following sequence: In the original mix, the effective volumes (column 2) are calculated from batch weights (column 1), and then the mix is adjusted by volume (column 3) to correct the cement content. Batch weights on a dry basis (column 4) are calculated from these volumes and then adjusted for changes in moisture content (column 5). The effective volume (column 6) is calculated to determine the water content. (Adapted from ACI 211.2-69. Reproduced by permission.)

The fine-aggregate content is changed to keep the yield constant on a dry-mix basis. When adjusting for stockpile moisture contents, the effective volumes of both the coarse and the fine aggregates change because the SGF changes. The water content is then adjusted to keep the yield constant. Because of the high sensitivity of aggregate weight to moisture content, it is important to closely monitor the unit weight and yield of lightweight concrete mixes at the ready-mix plant and job site to adjust for changes in aggregate moisture content.

Moderate-Strength Lightweight Concretes

This classification is intermediate between structural lightweight concrete and insulating concrete. Such concrete combines additional insulating capacity with some useful strength. Lightweight concrete block is a common application for this class of concrete. Typical aggregates are pumice, scoria, or expanded clay or shale. The aggregates are selected or manufactured with lower unit weights than are needed for structural lightweight concretes to provide better thermal insulation. "No-fines" concrete, using either normal-weight or lightweight aggregates, also fits into this category, but is discussed under gap-graded concrete.

Ultra-Lightweight Concretes

This classification includes those concretes with unit weights of less than 1100 kg/m^3 (70 lb/ft^3) and compressive strengths below about 7 MPa (1000 lb/in.^{2}). With ultra-lightweight concretes, strength requirements are minimal, and the concrete is used for its other properties, principally thermal insulation and light weight; such concretes are used primarily for insulating fills on roof decks, metal decking or floors, underground conduit linings, fire walls, and non-load-bearing fill over structural concrete. Thermal conductivity and compressive strength are dependent on concrete density, as shown in Figure 17.4 (Section 17.1) and Figure 20.6.

Vermiculite and Perlite Concretes In the United States, expanded (exfoliated) vermiculite and expanded perlite (Chapter 7) are most commonly used as aggregates in insulating concrete. The expanded materials are very durable and do not interact chemically with the cement paste, but the concretes have high water requirements because of the high absorption of these aggregates. Strength is dependent on the cement content and the use of normal-weight sand; rich mixes have strengths around 7 MPa (1000 lb/in.^{2}), but a greater density (Figure 20.6) and higher thermal conductivity than mixes with less cement. The concretes are made with a high slump and a high air content (25 to 30%) for easy handling and finishing. The high air contents prevent segregation, and the high slump means that generally only screeding and darbying are required to give a satisfactorily smooth surface. Since the thermal conductivity is strongly dependent on moisture content, insulating concretes should be allowed to dry after curing before being sealed. Insulating materials based on vermiculite and perlite are covered by ASTM C 196 and ASTM C 610, respectively.

Aerated Concretes The lightest kind of aggregate possible is, of course, air, and this is the basis of aerated or cellular concrete. The "aggregate" is a uniform cellular structure of air voids distributed throughout a matrix of cement paste or mortar, so that the term

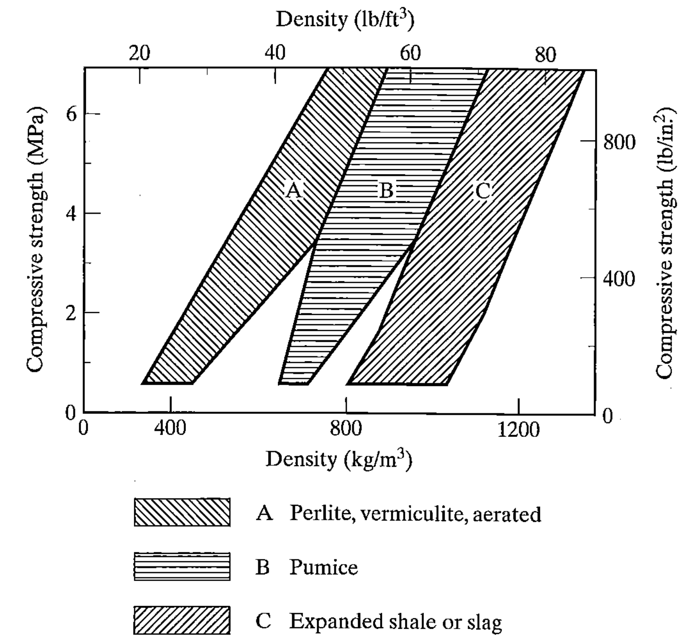


FIGURE 20.6

Influence of density on the strength of insulating concretes. [Adapted from S. H. Kosmatka, B. Kerkoff, and W. C. Panarese, *Design and Control of Concrete Mixtures*, 14th ed., Portland Cement Association, Skokie, IL (2002).]

"concrete" is not strictly correct. Aerated concretes are more popular in England and Europe than in the United States, where vermiculite and perlite concretes are more extensively used. Aerated concrete can be either formed in situ or precast; in the latter case, high-pressure steam curing can be used, which gives a stronger product with better dimensional stability.

The nature of the void structure is similar to that of entrained air (i.e., discrete, nearly spherical bubbles), except that the voids are one or two orders of magnitude larger in diameter (0.1–1 mm, 0.004–0.04 in.), large enough to be seen with the naked eye. Thus, the concrete will have good frost resistance, although it is of low strength. It also will not show rapid absorption of moisture, since the bubbles are not interconnected. The void system is formed either by generating a foam within the concrete during mixing or by blending a preformed foam with the paste or mortar. Preformed foams are generally made from liquid concentrates, based on hydrolyzed protein or synthetic detergents, which are agitated with compressed air. A foam generator supplies foam at a standard rate, and this is blended with a slurry of cement and water to provide the finished product. It is also possible to add the concentrate to the cement slurry in a high-speed mixer, which provides a high shearing action that will cause foaming. In either case, the foam must be sufficiently stable to remain intact during handling and until the concrete has hardened. The other method of generating a foam during mixing is to form a gas within the concrete by a chemical reaction (hence the term "gas-concrete," which is sometimes used). Although other chemicals have been proposed, the material generally used is a finely divided aluminum powder (sometimes powdered zinc is used). The aluminum reacts with the soluble alkalis in the cement slurry to generate small bubbles of hydrogen.

$$Al + 2OH^- + 2H_2O \rightarrow Al(OH)_4^- + H_2\uparrow \quad (20.3)$$

The rate and amount of hydrogen generation determines the amount and character of the air–void system.

Typical properties of insulating cellular concretes produced commercially are density, 300 to 1100 kg/m^3 (20 to 80 lb/ft^3); compressive strength, 0.3 to 7.0 MPa (50 to 1000 lb/in.^{2}); and thermal conductivity, 0.1 to 0.3 W/m · K. In most cases, these are low-density, low-strength materials suitable for nonstructural insulating concretes, but it is possible to produce moderate-strength lightweight concretes from aerated concrete. The higher-strength materials have a lower void content and use mortar as the matrix. Density and thermal conductivity are a function of unit weight (Figures 20.6 and 17.4) in the same range as perlite or vermiculite concretes. Since the "aggregate" is air, the concretes have a very low modulus of elasticity and high drying shrinkage.

20.2 HEAVYWEIGHT CONCRETES

Worldwide, heavyweight concrete is used most often as a shielding material to protect workers from the damaging effects of gamma rays and neutrons produced by radiation sources. Where space is not a limitation, concrete of normal density can be used most economically, but concrete of high density, >3200 kg/m^3 (>200 lb/ft^3), has better attenuating properties and can be used in thinner sections for the same amount of shielding. The desirable properties of heavyweight aggregates and the factors affecting attenuation of radiation by concrete are discussed in Chapters 7 and 17, respectively. In this section, we discuss the proportioning and properties of heavyweight concretes, in general, and the particular requirements of concrete for nuclear reactors.

Heavyweight concrete can be proportioned using the ACI method of absolute volumes developed for normal-weight concrete, as described in Chapter 10. Thus, it is necessary to determine the bulk specific gravities of both the fine and coarse aggregate, the fineness modulus of the fine aggregate, and the unit weight and maximum size of the coarse aggregate. Selection of quantities of cement, water, and coarse aggregate are the same as for normal-weight concrete. However, heavyweight aggregates tend to give a harsh mix because of their shape and texture, so that it is desirable to use fine aggregates with a lower fineness modulus or a higher ratio of fine to coarse aggregate. Cement contents are therefore generally quite high, greater than 350 kg/m^3 (600 lb/yd^3). This helps to improve the shielding characteristics of the concrete because of the high bound water content of the paste.

Heavyweight concrete can be pumped, but over much shorter distances than normal-weight concrete, because of its high density. Also, pressures on formwork are much greater, and formwork must be constructed accordingly. Because of the high density, segregation can be more pronounced, and it is recommended that both fine and coarse aggregates be high-density materials to minimize undue segregation. The preplaced aggregate method is often used to avoid problems of segregation and to produce concrete of uniform density. Preplacing the aggregate may also assist in difficult placements where reinforcement is rather congested or where there are many items to be embedded in the concrete.

The properties of heavyweight concrete are similar to those of normal-weight concrete. Strengths can be estimated from the same *w/cm* ratio relationship, with the exception that compressive strength of preplaced concrete made with steel punchings or smooth cubical pieces rarely exceeds 21 MPa (3000 psi). Preplaced concrete made with sheared reinforcing bars will produce concrete of normal strength. The physical properties of heavyweight concrete will depend to a large extent on the properties of the aggregate. Thermal conductivity is about the same as for normal-weight concrete when magnetite is used, but is up to 50% lower when barite is the aggregate; it is higher if steel aggregate is used.

Concrete for Nuclear Reactors The use of prestressed concrete reactor pressure vessels (PCRV) for nuclear power generation places much greater demands on heavyweight concrete than other forms of radiation shielding. Pressure vessels operate at higher temperatures and stress levels than conventional structures, and the concrete may be under appreciable thermal and moisture gradients. The concrete must maintain its integrity in both the short and long terms under these more severe operating conditions, so that the thermal properties of the concrete are an important consideration in design.

PCRVs can be expected to operate at bulk vessel temperatures of about 70°C (160°F) under relatively high multiaxial stresses, and the vessel will be subjected to some thermal cycling during its lifetime. Inelastic deformations, such as creep and shrinkage, should be minimized since they can cause microcracking of the concrete and loss of prestress. Similarly, thermal incompatibility between aggregate and cement paste can also cause microcracking within the concrete, particularly if temperature cycling occurs. Microcracking not only reduces the predictability of vessel response to prestress and pressure forces, but also reduces concrete strength and increases permeability.

In many cases, knowledge of the appropriate concrete properties under rather complex conditions is insufficient for proper analysis, and much research has been directed toward learning more about the behavior of concrete under multiaxial stresses at elevated temperatures and conditions of temperature cycling. However, enough is known about the properties of concrete materials to make informed choices and to recognize the compromises that have to be made. These considerations are summarized in Table 20.10.

Concrete strengths should be higher than for conventional concrete to ensure adequate long-term sustained strength. For example, it has been suggested that a compressive cylinder strength of 45 MPa (6500 lb/in.^{2}) would correspond to a long-term structural strength of 21 MPa (3000 lb/in.^{2}). A high strength requirement means that the use of low *w/cm* ratios is desirable, but high workabilities are required to place and consolidate the concrete around high concentrations of reinforcement. Therefore, a high cement content is needed, and this can possibly result in thermal cracking if steps are not taken to control the heat of hydration. High cement contents can also lead to larger creep and shrinkage strains than would be desirable. These can be minimized by the use of aggregates with a high modulus of elasticity to restrain paste deformations.

A low thermal expansion concrete is desired to minimize thermal stresses. On the other hand, stress losses that can occur on heating to typical working temperatures are

TABLE 20.10 Material Properties for Concrete for Nuclear Reactors

Material		*Concrete Properties*
Cement	High content	Good workability
	Low content	Low creep and shrinkage, low heat of hydration
Water	Low content	High strength, low creep and shrinkage, low permeability
	High content	Good workability, good shielding
Aggregate	High density	Good shielding
	High modulus	Low creep and shrinkage
	High thermal expansion	Low internal stresses
	Low thermal expansion	Low thermal expansion

believed to be due primarily to thermal incompatibility between the coarse aggregate and the cement paste, leading to bond failure. Using an aggregate with a high siliceous content increases the thermal coefficient of the aggregate so that it more nearly matches that of the paste, but this increases the thermal expansion of the concrete.

Moisture loss in service is also of considerable importance in reactor design, since it can lead to differential shrinkage strains and the generation of internal stresses. Loss of moisture also decreases the thermal conductivity of the concrete and thus may aggravate thermal stresses during temperature cycling. Furthermore, the water content of the concrete affects its shielding capabilities, since water is a good attenuator of slow neutrons. In the cases where hydrous aggregate has been used to improve neutron-shielding properties, the maximum service temperature must be limited to avoid dehydration of the aggregate.

The use of low *w/cm* ratios to attain high strength is also beneficial in producing concrete of low permeability (assuming good compaction). This is advantageous in minimizing leakage in the event of failure of the impermeable liner that is used in pressure vessels. Low permeability also reduces the rate of moisture loss, thereby reducing shrinkage gradients.

The early use of concrete in prestressed nuclear reactor pressure vessels drew on the limits of knowledge about concrete. Subsequent research and field evaluation of operating vessels has greatly expanded the knowledge base, but considerably more understanding of the material is needed. The use of heavyweight concrete in reactor pressure vessels is a very interesting materials problem, requiring a considerable range of service requirements. The reduction in use of nuclear power in many parts of the globe, beginning in the 1980s, however, has slowed study of this field.

20.3 ARCHITECTURAL CONCRETE

One of the advantages of concrete is its versatility not only as a building material, but also as an architectural material. Concrete can be cast into a variety of complex shapes; it can be given a variety of special surface finishes, textures, and colors; and it can be

used in sculptures, murals, and other special aesthetic creations. Architectural effects can be obtained by a suitable choice of concrete materials, formwork materials, special casting techniques, or texturing of hardened concrete surfaces. Generally, architectural concrete will cost more than plain structural concrete, but may actually be cheaper if other architectural treatments, such as painting or brick cladding, are included in the comparison. Since careful control of color, texture, and design details is more important for the overall aesthetic effect, it is important that special attention be paid to uniformity of the component materials and of the concrete, and that strict control over fabrication processes be maintained. These special considerations mean that better overall quality control is necessary; this will result in a better overall product and improved properties for a maintenance-free structure.

It is not possible to do justice in the space available to the infinite variety of aesthetic effects that concrete can offer. Thus, this section will only survey the various materials and processes that have been used to produce special effects.

Colored Concrete

Manipulation of color is one strategy an architect can use to advantage. Color in concrete can be achieved through use of special cements and pigments or by the selection of colored aggregates. Construction costs of white or colored concretes are higher than for conventional concrete because special attention must be paid to handling and consolidation to attain uniform colors. This is discussed in the next section.

Cements The gray color of portland cement may not set off the color of aggregates to advantage and will modify the colors obtained by the coloring agents. Therefore, white cement (see Chapter 3) is commonly used, even though it is considerably more expensive than ordinary portland cement. White cements will produce colored concretes with brighter, truer colors and can be used to produce pastel shades as well as white concrete. However, because of atmospheric pollution, white or colored concrete (particularly light colors) may soon become discolored and may need periodic sand blasting or other cleaning to restore the original color. Some cements may be buff, tan, or light gray in color or have a greenish hue, depending on the nature of the raw materials available.

Coloring Agents The most popular means of obtaining integrally colored concrete is to add color pigments during mixing. However, special care must be taken to ensure good dispersion of the pigment within the concrete. Some cement companies provide a range of colored cements by intergrinding a pigment with a white cement at the factory. Good control of color uniformity can thus be obtained. Even so, it is difficult to make successive batches of concrete over a period of time with exactly the same color. This is one of the reasons why colored concrete is not often used. Pigments should provide a permanent color that will not fade on normal exposure to the elements. They should be particularly resistant to the alkaline effects of the lime, which can cause some pigments to lose their color. Typical pigments are listed in Table 20.11.

Color pigments are finely divided solids that can be classified as mineral admixtures, although their levels of addition are much lower than those of other mineral admixtures. They can be used with other admixtures; indeed, the use of a water-reducing

TABLE 20.11 Typical Pigments for Integrally Colored Concrete

Color	*Pigment*	*Typical Dose*[a] *(by Weight of Cement)*
Red	Iron oxide (hematite)	5%
Yellow	Iron oxide	5%
Green	Chromium oxide	6%
Blue	Phthalocyanine	0.7%
	Cobalt oxide	5%
Black	Carbon	2%
	Iron oxide (magnetite)	5%
Brown	Iron oxide	5%

[a] Amount above which no increase in intensity is observed.

admixture is often advisable since it can improve the dispersion of the pigment and minimize its tendency to float to the surface, as well as reducing laitance and efflorescence, which will reduce color intensity. The addition of calcium chloride should be avoided since it can interfere with color uniformity and indeed often discolors even nonarchitectural concrete. In some cases, colored admixtures are available that are carefully controlled blends of pigments with compatible admixtures. Pigments for integrally colored concrete are specified under ASTM C 979.

Pigments can also be incorporated in products that are applied directly to horizontal surfaces. They can be used during the finishing process as a dry-shake application to provide color at the surface only, rather than through the complete section. They can also be formulated with surface hardeners, abrasion-resistant toppings, and so on, that are used to treat horizontal wearing surfaces. A problem with this approach to coloring is obtaining uniform color and properly finished concrete. However, this is a more economical use of pigments, and greater intensity of color can be more easily achieved.

Chemical stains have also been used to color concrete surfaces. These materials are aqueous, acidic solutions of metallic salts. The salts react with the alkaline concrete to deposit metal hydroxides (or hydroxyl salts) within the pores. The colors produced depend on the nature of the metal salt, but are usually black, green, or various shades of brown. The acid etches the concrete slightly to assist in penetration of the stain. Generally, only the cement paste is stained, the aggregate being unaffected. Thus, stains can be used to advantage with exposed aggregate concrete or other textured surfaces where integrally colored concrete is not desired.

Aggregates There are a wide variety of naturally colored rocks that can be used as concrete aggregates to give pleasing color effects. The range of colors is much more varied than can be obtained by the use of pigments. The kinds of rock that can be obtained in colored forms are very widespread, and some variety of color can be obtained almost anywhere. Most common are the brown and ochre shades that can be obtained in many river gravels and the pure white of quartz. The latter is particularly useful as a fine aggregate (silica sand) when white or colored concrete is used. Marbles offer the widest range of color, while granite (which is an excellent concrete aggregate) is obtained in shades of pink, gray, black, and white. The range of natural colors can be extended by the use of colored glasses and ceramics; however, many glasses are susceptible to the alkali–aggregate reaction. Colored aggregates are best used to advantage in exposed aggregate finishes or in sawn and polished terrazzo slabs. They are an expensive form of aggregate since the material must be carefully processed to give a uniform color and texture.

Mix Design and Handling

Mix design, choice of formwork materials, handling and compaction, and curing methods can all lead to color and textural variations in form-finished surfaces. The major problems are summarized in Table 20.12.

TABLE 20.12 Surface Blemishes on Form-Finished Concrete

Description	*Most Probable Causes*
Color variation	Variations in materials Incomplete mixing Segregation during placing Variations in batching
Hydration discoloration	Variable absorption of formwork Formwork leakage Uneven application of form release agent Uneven curing
Aggregate transparency	Low sand content Smooth, flexible formwork Excessive vibration External vibration
Dye discoloration	Stains or dyes on form face Reactions with form coatings Impure release agent
Brown discoloration	Impure release agent Excessive amounts of release agent Rust from forms
Honeycombing	Insufficient fines Compaction inadequate Leaking joints
Bugholes	Oversanded mix Nonabsorptive forms Poor wetting characteristics of form face Inadequate vibration
Spalling or chipping	Low-strength concrete Insufficient form release agent Forms removed too early
Crazing	Mix too rich; *w/c* ratio high Formwork with low absorbency Insufficient curing Forms removed too early

Mix Design The principles of mix design for architectural concrete are no different from ordinary structural concrete. When the surface effects call for intricate as-cast finishes, particular attention must be paid to workability under vibration to ensure the intimate contact with forms that will give sharp, clean details. A maximum *w/cm* ratio of 0.46 is recommended. The slump should be as low as possible, consistent with the materials and type of placement, to reduce the possibility of bleeding, which can lead to color variations within a lift. To obtain surfaces of uniform color, a high cement content and additional fine sand are desirable. Where exposed aggregate finishes will be used, a higher-than-normal proportion of coarse aggregate is desirable to give a good density of exposed aggregate. The use of gap-graded aggregate with a narrow size range can provide a more uniform distribution of exposed aggregate. Fine sands, rather than coarse sands, should be used to maintain workability.

Mixing, Transporting, Placing, and Compacting Proper handling of concrete is critical to ensure high-quality concrete finished free of defects and blemishes. This is particularly true when white cement or pastel colored cements are used, because they will clearly reveal differences in texture, surface contamination, and the presence of impurities. Accurate batching is as important for aesthetic considerations as it is for structural purposes. Variations in water and cement content can have a considerable effect on the color of the concrete. A low *w/cm* ratio gives a darker color, due in a large part to the fact that the ferrite phase hydrates to a lesser extent in these areas. A color standard at the batch plant and job site is a useful tool to help ensure consistency. Equipment and conveyances should be kept clean and well maintained to prevent contamination by oil and grease. Mixing should always be exactly the same for each batch when color is used to ensure the same dispersion of pigment. Cement and pigment should be preblended, and mixing times may be longer than usual to ensure uniform dispersion.

Concrete should be placed as close as possible to its final position to minimize potential segregation. It should not be allowed to splatter the formwork, since this can cause surface defects. When as-cast finishes are required, consolidation is an important consideration. There is more tolerance when surfaces will later be textured, but even in such cases gross variations in consolidation may not be masked. Proper vibration is critical for a uniform finish that is free of blemishes and has clean, sharp edges. Chapter 11 describes the process of vibration and the correct procedures to be used for good vibration. When a well designed low-slump concrete is used, it is unlikely that ill effects will be caused by overvibration; undervibration is more likely to cause surface blemishes. If vibration is not performed properly, air driven to the surface of the forms will not be fully expelled. The vibrator must not be allowed to touch the forms, however, as this will result in areas rich in paste, which will cause textural and color variations. When exposed aggregate finishes are to be used, the vibrator should be kept at least 75 mm (3 in.) from the form face to prevent pockets of fine materials from forming.

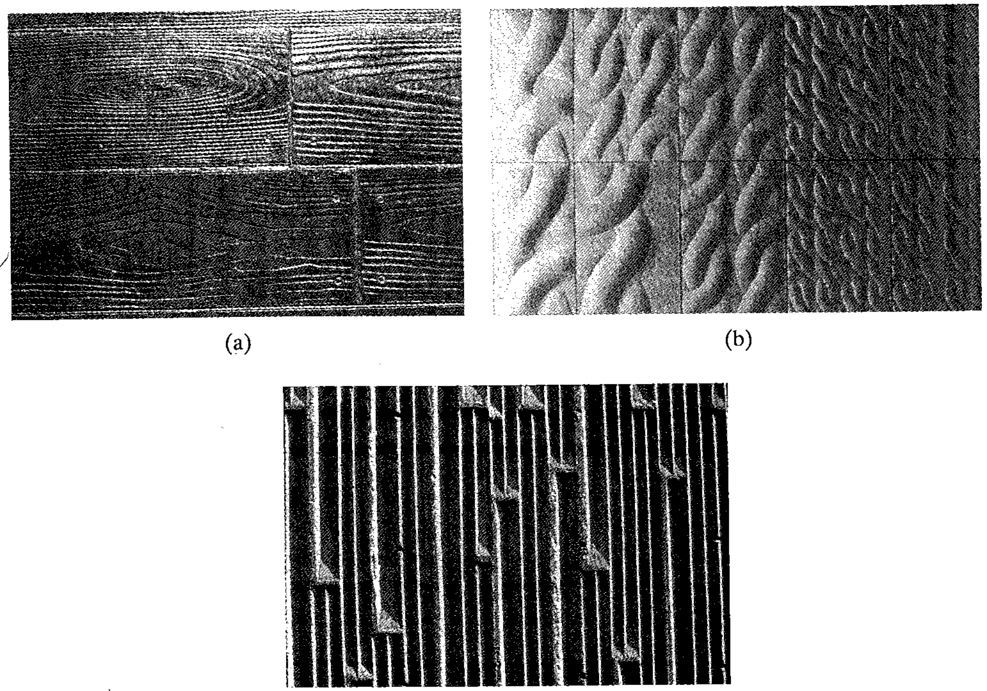
Formwork If an as-cast finish is required, special care must be taken in choosing materials (see Table 20.13) and in the construction and preparation of formwork. Concrete reproduces the surface against which it is cast very faithfully. This can be used to advantage to obtain wood-grain effects by casting directly against wood (Figure 20.7) or to obtain textures imprinted in rubber or plastic form liners. However, concrete will also faithfully reproduce blemishes and defects on the form surfaces, such as knots, patches, and marks stamped or drawn on the surface. Staining may be caused by untreated wood, by form oils, or by rusting steel. Certain plywoods and fiberboards can cause severe staining, and protective coatings may be needed. Staining is particularly noticeable with white concrete, and special nonstaining release agents are available.

If the formwork is absorbent (e.g., wood), it will remove water from the concrete before setting. The resultant moisture movement brings cement particles to the surface. Thus, at areas of higher absorption there will be a low *w/cm* ratio at the surface, which causes a darker coloration, as discussed earlier. This is known as *hydration discoloration* and can be eliminated by sealing the form surface, by using a material of uniform absorbency (not easy), or by choosing a nonabsorbent form material. Plastic sheeting can be used as a nonabsorptive form liner, but thin plastic film should not be used because wrinkles are not easily eliminated and will be reproduced on the surface. Concrete cast against nonabsorbent surfaces has a lighter color. Also, materials of low absorbency tend to increase the number and size of "bugholes" (called "blow-holes" in some countries). These are small entrapped air voids that occur at the form surface and are difficult to remove completely even with adequate vibration. Bugholes are reduced by decreasing the amount of sand, particularly the coarser fractions, but low sand contents lead to greater color variations. Thus, some trade-off is needed between uniform color and the occurrence of bugholes. Form release agents may encourage the formation of bugholes, and the incorporation of a surface-active agent is recommended.

Aggregate transparency occurs when concrete is cast against smooth nonabsorbent forms. A very thin layer of mortar always separates the coarse aggregate from the form. When in contact with a smooth surface, the calcium hydroxide crystals that form during hydration are oriented to form a transparent coating through which the

TABLE 20.13 Formwork Materials for Architectural Concrete

Form Material	*Comments*	*Number of Times Can Be Reused*
Plywood	Variable absorption may cause staining: coat with wood sealer	Up to 5
Plastic-coated plywood	Joints must be sealed, may cause staining	10
Steel	May cause variations in color and textures; rust may cause stains: coat with epoxies	50–100
Fiberglass		20–30
Form liners:		
Lumber	Special textures; variable absorption, and may retard set: coat with wood sealer	1–20
Rubber, PVC	Special textures; flexible; uniform surfaces	100
Fiberglass	Special shapes and textures	20–30



(a) (b) (c)

FIGURE 20.7

Examples of surface texture provided by formwork; (a) wood grain; (b) rubber mat; (c) plastic liner. (Photographs courtesy of Portland Cement Association.)

aggregate is visible. This gives a mottled appearance. On rough surfaces or those in contact with absorptive forms, calcium hydroxide crystallizes randomly and forms a translucent layer. Aggregate transparency can be corrected by roughening the surface by light etching or abrading, or by using an absorptive form liner. Some release agents give a roughened surface.

Tightly constructed formwork that will not leak is important for good finishes. Loss of water at leaky joints will lead to hydration discoloration, while loss of paste gives changes in texture and color. Excessive loss of paste can lead to honeycombing and scabbing (Figure 20.8)—a situation that should be avoided even with structural concrete. Leakage at joints can also aggravate the bughole problem, as paste is expelled and air sucked in during the vibration cycle (see Figure 20.9). Whenever possible, joints should be sealed with tape, filled with a suitable caulking material, or protected by compressible gaskets.

Finishing and Flatwork Troweling of white or colored concrete flatwork improves color uniformity by smoothing out minor differences in texture and bringing more cement (and pigment) to the surface. This results in better color and a more reflective

surface. However, if troweling is delayed until the concrete is too stiff, "trowel burning" may occur. This is discoloration caused mostly by uneven compaction. To avoid trowel burning, concrete surfaces should be protected to eliminate excessive evaporation, calcium chloride should be prohibited, and plastic trowels should be used.

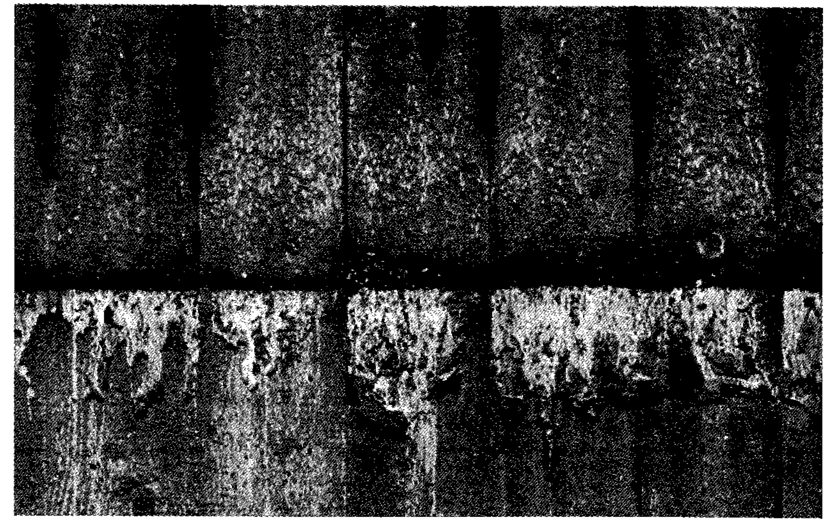


FIGURE 20.8

Effect of leakage of formwork joints. (Photograph courtesy of Concrete Construction Publications. Produced herein with permission. Copyrighted 2001 by Hanley–Wood, LLC, 426 South Westgate St., Addison, IL 60101.)

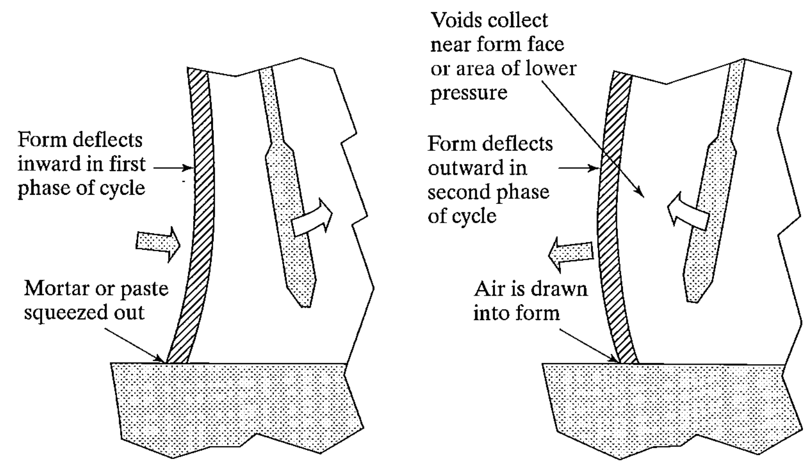


FIGURE 20.9

Effect of vibration on leaking formwork joints (movements and deflections exaggerated for emphasis). [From *Concrete Construction*, Vol. 17, No. 11, pp. 536–538 (1972). Produced herein with permission. Copyrighted 2001 by Hanley–Wood, LLC, 426 South Westgate St., Addison, IL 60101.]

There are many other factors that can lead to mottling or discoloration of concrete flatwork. For example, the alkali content of the cement, or inappropriate curing procedures, can cause or aggravate discoloration. The mottled appearance of a surface can be caused by changes in the amount of hydration of the ferrite phase or by local deposition of salts within the pores. In some cases, repeated washings may remove the discoloration; in other cases, however, chemical treatment may be necessary.

Curing Good curing practices are important for maintaining uniformity of concrete surfaces as well as for ensuring proper hydration and strength development. Concrete that is allowed to dry prematurely is lighter in color than concrete that is cured longer. This is the same effect as hydration discoloration, since insufficiently curing cement paste has the same effect as increasing the *w/cm* ratio. Thus, concrete should be uniformly cured, or color variations will again result from hydration discoloration. If plastic sheeting is used to keep concrete moist, partial contact with the concrete should be avoided, since this can cause mottling. Waterproofed paper should not cause staining of surfaces. Concrete is best cured for several days before removing formwork to ensure uniform hydration and good strength development, to reduce possible damage during formwork removal, and to minimize cracking or crazing caused by rapid drying. Curing compounds are not recommended, since they can cause staining or discoloration.

Concrete sealers are sometimes used to protect white or colored concretes from contamination by atmospheric pollution. There is a wide variation in the performance of such materials: Some can cause permanent discoloration of the concrete, and they are generally avoided except in areas of heavy pollution.

General Cleanup and Patching After removal of formwork, the exposed concrete should be cleaned and restored to a uniform, smooth surface. This requires packing bolt holes and tierod holes with mortar, repairing honeycombing, removing bulges and projections, removing stains, and filling in bugholes and air voids at the surface. These repairs should be made as soon as possible after form removal. If concrete placement and consolidation have been carefully controlled, there should be a minimum amount of cleanup needed, but in extreme cases, considerable amounts of poor-quality concrete may need to be removed and replaced. Small holes and cavities should be packed with mortar, as dry as possible, and projections removed by chipping and tooling, and rubbed smooth. Small areas of honeycombing can be grouted, but large areas should be entirely removed and replaced. Staining generally occurs from form-release agents or rust marks that are most easily removed by light sandblasting, since they are only surface problems. The surface should then be wetted down, and a grout of portland cement and fine sand should be applied over the whole surface by brushing, spraying, or rubbing. This fills up small air holes and covers up patches and other nonuniformities of the surface texture and appearance. The grout is well floated, and the excess scraped off. After the surface has dried, it is rubbed with burlap to remove any remaining dried grout. If the dried grout is left too long (e.g., overnight), it is too difficult to remove. Done properly, this treatment will result in a smooth, unblemished surface, uniform in color. It is important that exactly the same materials and proportions should be used throughout the patching operations. Trial mixes of the grout should be made ahead of

time to get the exact color desired. Similarly, any localized patching, such as repair of honeycombing, should use the same materials and proportions used initially in order to get a good color match.

Textured Concrete Surfaces

Although color plays an important role in architectural concrete, the provision of texture and design is probably even more important. Textured surfaces are not only pleasing in themselves, but also eliminate or mask small variations in surface texture and color on the form finish surface. Details of design are beyond the scope of this book, but it is appropriate to discuss ways of achieving textured finishes. There are three main approaches to this objective: textured form liners, exposed aggregate finishes, and mechanical finishes (the latter is discussed in Chapter 11).

Textured Form Liners The use of textured form liners can produce some interesting effects (Figure 20.10). These can range from ridging or dimpling, through simple repetitive geometrical patterns, to complex mural designs. Smooth, flexible liners can be molded to give a variety of textural patterns. They are generally made from rubbers, thermoplastics (such as PVC), or fiberglass; can be reused many times; and are suitable for both precast and cast-in-place concretes. Special designs for murals are sculptured either in sand beds or in expanded polystyrene blocks. Panels are generally cast face downward; this helps to obtain a clearer reproduction of the details of the design. Exposed aggregates or specially embedded materials can be incorporated into such

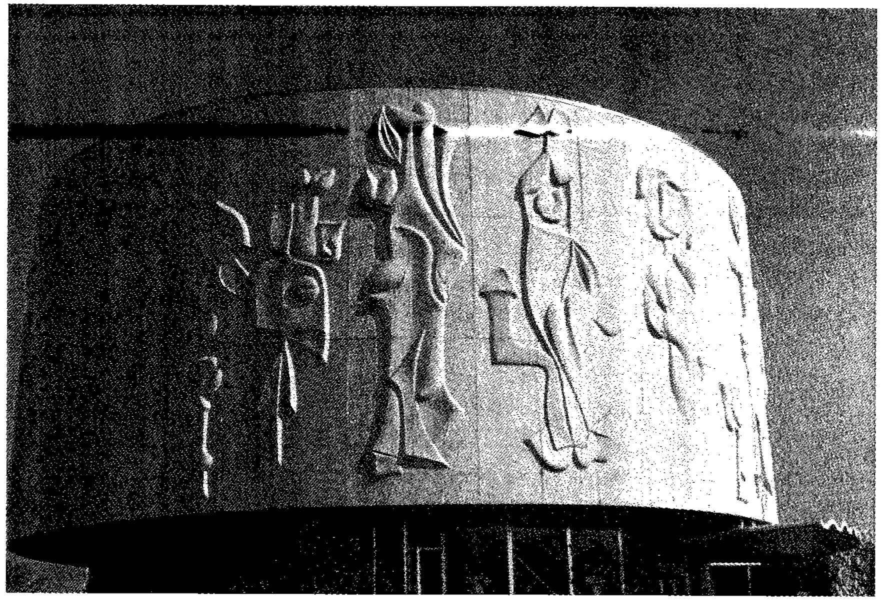


FIGURE 20.10
Example of concrete mural design. (Photograph courtesy of Portland Cement Association.)

designs, leading to an infinite variety of artistic effects. Mechanical texturing can also be used subsequently.

Exposed Aggregate Finishes Exposing coarse aggregate at the concrete surfaces uses to advantage the variety of color, shape, and texture of the material. Exposure even of ordinary gravel or crushed stone can give pleasing effects. There are several methods that are used to obtain exposed aggregate finishes.

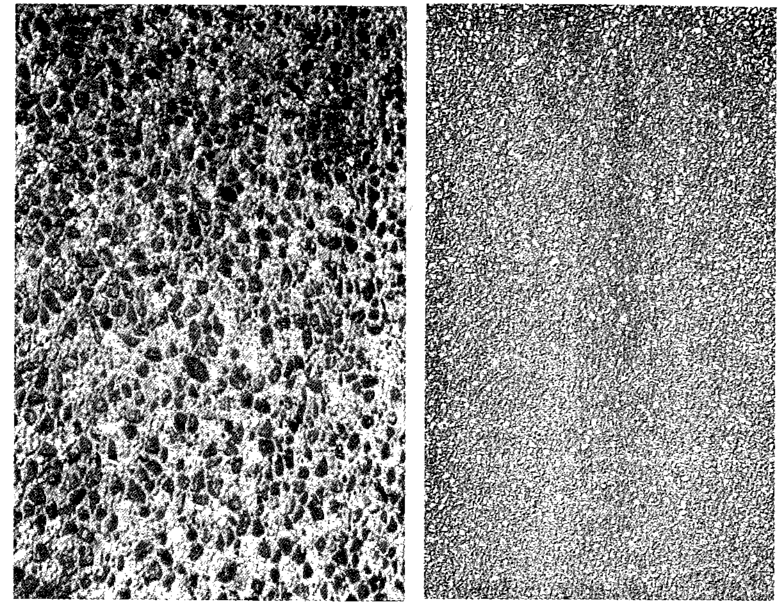
The *face-up method* is widely used for patios and sidewalks and is a variation of the dry-shake method for producing special surfaces. A base of ordinary concrete is cast, screeded, and troweled flat. The surface is covered uniformly with the desired aggregate, which is then floated into the surface and again troweled flat. Just about the time of initial set, a thin layer of cement paste is carefully removed by washing with a light spray of water. High-pressure water jets have also been used, but require skill and experience for satisfactory results. Alternatively, a retarding admixture is applied to the surface of the concrete after troweling. About 16 h later, when the rest of the concrete has set, the retarded surface paste can be brushed and washed off easily. Special toppings can be cast on top of plain concrete using colored aggregates and white or colored cements. Large pieces of aggregates can be hand-laid in special patterns.

A retarder can also be used as a form coating to retard the concrete in contact with the form. The retarder can be painted on directly, or cloth impregnated with retarder can be used to line the form. Special admixture formulations are available for exposed aggregate finishes. Both horizontal and vertical surfaces can be cast by this method. Horizontal casting is used in precast work and allows greater versatility. The panels are cast *face down* with the retarding form at the bottom. As in the face-up method, a special facing mix may be used that can consist of aggregates specially selected for color, shape, and texture. A greater concentration of coarse aggregate is possible, and white or colored cement can also be used. Plain concrete is then cast on top to form an integral panel. This approach is not possible in vertical casting; the aggregate to be exposed must be used throughout the concrete. The prepacked aggregate method can be used in vertical casting.

The kind and amount of retarder depends on the depth of exposure. A water-insoluble retarder is recommended for retardation to a depth of 1.5 to 3 mm (1/16 to 1/8 in.), while water-soluble retarders penetrate more deeply into the concrete and will give greater exposure. The depth of exposure should not exceed one-third of the average diameter of particles. The choice of aggregate size is thus dependent in part on the depth of texture (see Figure 20.11), which in turn depends on the distance at which the texture should be visible (see Table 20.14) and the total area cast.

For only slight exposure (less than 3 mm or 1/8 in.), sandblasting or acid etching can also be used. Sandblasting requires special equipment and is expensive. Also, uniformity of finish is difficult to achieve, while softer aggregates may be abraded sufficiently to lose their intensity of color. Light sandblasting can be used to minimize minor color variations, such as light mottling, or slight changes in color from lift to lift. Acid etching is designed to dissolve away hardened paste at the surface, but is slow and difficult to control uniformly. It cannot be used with carbonate aggregates (limestones, dolomites, marbles), because the aggregates are attacked by the acid, which may result in discoloration or loss of integrity. A light acid etch is sometimes used to clean an exposed aggregate surface. The use of acid is hazardous and proper protective clothing should be worn.

The *sand-bedding technique* is used to advantage when exposing large aggregates or when special effects are required, such as a flat-stone arrangement. A bed of sand, the same as used in the concrete, is laid down and the aggregate embedded in it to the required depth. The whole is then damped down for consolidation, and structural concrete is cast on top. After the concrete has hardened, the panel is removed and excess sand is washed off the face. This method allows a greater depth of exposure—12 to 50 mm (1/2 to 2 in.)—which is more than is possible with chemical retarders.



(a) (b)

FIGURE 20.11

Examples of exposed aggregate finishes. (Photographs courtesy of Portland Cement Association.)

TABLE 20.14 Visibility Scale for Exposed Aggregate Concrete

Size of Exposed Aggregate	Distance at Which Texture Is Visible
6–12 mm (1/4–1/2 in.)	6–9 m (20–30 ft)
12–25 mm (1/2–1 in.)	12–22 m (40–75 ft)
25–50 mm (1–2 in.)	22–40 m (75–125 ft)
50–75 mm (2–3 in.)	40–55 m (125–175 ft)

20.4 OTHER TYPES OF CONCRETE

Gap-Graded Concrete

In Chapter 7, during the discussion of aggregate, mention was made of gap grading, in which certain aggregate sizes are omitted. Gap-graded concrete is not inherently better than concrete made with continuously graded aggregate. Gap grading of coarse aggregate gives better exposed aggregate finishes. It will improve the workability of undersanded mixes, but is still susceptible to segregation. In fact, segregation during handling and consolidation is a major drawback to gap-graded concrete, and special care must be taken in mix design and in handling techniques.

"No-Fines" Concrete A special type of gap-graded concrete is "no-fines" concrete, where the fine aggregate is omitted entirely and a uniform size of coarse aggregate is used. This type of concrete can be viewed as an assembly of coarse aggregate particles cemented together by a layer of hardened cement paste at their points of contact. The material has an open structure with a high void content and a fairly low cement content. Thus, no-fines concrete has a low unit weight, low strength, low shrinkage, and a low thermal conductivity (Table 20.15). It is suited for applications where strength requirements are not great, but lightweight and insulating properties are of particular importance. When the aggregate is itself a lightweight material, it has excellent insulating properties and is an ultralightweight material.

Since no-fines concrete is very porous, it is highly permeable to air and water and thus should not be used in foundations unless an effective moisture barrier is provided. However, there is little tendency for water to be drawn through in the absence of a pressure head (e.g., in an exterior wall exposed to rain), because there is very little capillary action. Water can only pass through by vapor transmission, and moisture penetration is only two to three times the diameter of the aggregate particles under conditions of high humidity and no air movement.

When designing no-fines concrete the paste content should be just sufficient to thoroughly coat each aggregate particle. The *w/cm* ratio should not be too high, or the

TABLE 20.15 Properties of No-Fines Concrete

a/c Ratio	*w/c Ratio*	*Cement Content kg/m³ (lb/yd³)*	*Unit Weight kg/m³ (lb/ft³)*	*Compressive Strength, 28 days MPa (lb/in.²)*	*Thermal Conductivity (W/m · K) (Btu/ft · h · °F)*	*Shrinkage (%)*
6:1	0.38	259 (432)	2000 (125)	14.3 (2070)	—	—
8:1	0.41	193 (322)	1915 (120)	9.4 (1400)	—	0.018
10:1	0.45	155 (258)	1860 (116)	7.0 (1000)	0.74 (0.45)	0.019
Conventional Concrete						
6:1	0.40	250 (417)	2550 (159)	35 (5000)	1–4 (0.6–2.4)	0.035

paste will separate from the aggregate. When properly designed, the mix will not easily segregate during handling. It should be placed with light rodding; vibration or ramming should not be used. In spite of the high porosity of no-fines concrete, it must be air-entrained for frost resistance.

No-Slump Concrete

In a wide variety of applications, very stiff concretes that have virtually no slump, and thus have lower paste requirements, are selected based on economy or construction requirements, or both. These concretes have poor workability when consolidated by hand rodding, but can be compacted quite satisfactorily using high-frequency vibration.

Since the slump test cannot differentiate between different no-slump concretes, other workability tests must be used. The ACI Guide for Selecting Proportions for No-Slump Concrete (ACI 211.3R) suggests the use of the Vebe apparatus, the compacting factor apparatus, or the Thaulow drop table (Table 20.16). These tests are described in Chapter 9. The Vebe and drop-table tests more closely measure the behavior of no-slump concrete under vibration, which is an appropriate feature since heavy vibration is needed for good compaction. No-slump concretes are frequently used in precasting operations, where heavy vibration can be more easily provided, but they are also used for cast-in-place concrete. They are particularly suited for slipforming operations, such as slipformed pavements, where the concrete must maintain its shape unassisted before setting occurs. Another prime example is roller-compacted concrete, as described in Chapter 11. The proportioning of no-slump concretes, described next, is designed for cast-in-place concrete.

Proportioning The procedure described in the ACI 211.3R is basically the method of absolute volumes used for concretes of normal consistency modified to produce the stiff concretes. The reduction in workability is obtained by lowering the amount of water used and increasing the proportion of coarse aggregate (Table 20.17). The *w/cm*

TABLE 20.16 Workability of No-Slump Concretes[a]

Description	*Slump mm (in.)*	*Vebe (s)*	*Compacting Factor*	*Drop Table (rev)*
No slump				
Extremely dry	—	32–18	—	112–56
Very stiff	—	18–10	0.70	56–28
Stiff	0–25 (0–1)	10–5	0.75	28–14
Plastic				
Stiff plastic	25–50 (1–2)	5–3	0.85	14–7
Plastic	75–100 (3–4)	3–0	0.90	7–0
Flowing (very wet)	150–175 (6–7)	—	0.95	—

[a]Adapted from ACI 211.3R; reproduced with permission.

TABLE 20.17 Approximate Proportions of Water and Coarse Aggregate for No-Slump Concrete[a]

Consistency	Relative Water Content (%)[b]	Relative Coarse Aggregate Content (%)[b] 9.5 mm (3/8 in.)	12.5 mm (1/2 in.)	19 mm (3/4 in.)	25 mm (1 in.)	37.5 mm (1 1/2 in.)
Extremely dry	78	190	170	145	140	130
Very stiff	83	160	145	130	125	125
Stiff	88	135	130	115	115	120
Stiff plastic	100	108	106	104	106	109

[a] Adapted from ACI 211.3R; reproduced with permission.
[b] Relative to the amount of material required per m^3 (yd^3) to attain a slump of 75–100 mm (3–4 in.) according to recommendations in ACI 211.3R.

ratio is chosen on the basis of strength or durability requirements. The criteria for strength and durability are the same as for normal concrete (see Tables 10.3 and 10.4, Section 10.3). The method of calculation of batch weights is otherwise identical to normal proportioning.

Concretes for Mass Structures

We have from time to time discussed the special requirements of mass concrete. It is appropriate in this section to summarize the problems associated with the placing of mass concrete and the changes in material properties that can be tolerated. Since, historically, the majority of mass concrete is placed in dams, it is convenient to discuss this topic first.

Mass Concrete in Dams In dams, the strength requirements for the structure as a whole are not high. This can be used to advantage, because it is generally possible to make use of a local aggregate source even when it is not of high quality. Aggregate durability problems will not be a major concern in the core of the dam, with the exception of the alkali–aggregate reaction. Large aggregates, up to 150 mm (6 in.) in diameter, are used by the U.S. Bureau of Reclamation. The use of large aggregates requires the use of higher-capacity equipment than is normally employed in placing concrete. In some cases, very large stones or "plums" [0.03 to 0.5 m^3 (1 to 14 ft^3)] have been individually placed within the concrete mass, representing 20 to 30% of the total volume. Plums are no longer used since the savings in material costs are more than offset by extra labor costs and the effect on the heat generated is relatively small if 150-mm (6-in.) diameter aggregate is used in the concrete.

Large aggregate sizes allow quite low cement contents (less than 150 kg/m^3, 250 lb/yd^3) to be used in the interior of a dam. This helps to reduce the heat of hydration, which can be further reduced by the use of a Type IV cement or a pozzolan as a replacement for some of the cement. Internal heating of the structure is also reduced if the concrete temperature is kept below 16°C (60°F) at the time of placing. Even so, it is usually necessary to cool the concrete during curing by pumping water through a system of pipes embedded in the concrete at the top of each lift. Thermal cracking occurs

not on heating, but on subsequent cooling, when thermal stresses become greatest. When heat generation is controlled, the proper provision of contraction joints should prevent random cracking. The joints may be filled with a waterproof joint sealant or grouted when the structure has cooled.

More care should be taken in the quality of concrete used for exposed faces. Concretes should have higher cement contents, lower *w/cm* ratios, and good-quality aggregates to ensure adequate durability. Air entrainment will be needed for frost resistance, and on spillway sections, special measures for improved abrasion and cavitation resistance are required. Special care should be taken to prevent excessive moisture loss or temperature changes at the surfaces after the concrete has been newly placed, to avoid the possibility of surface cracking.

Mass Concrete for Other Structures Although dams represent the most obvious application of mass concrete, the term applies to any structure "with dimensions that are large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking."[1] This means that many of the precautions applied in dam construction must be applied for structural members with minimum dimensions of 0.9 m (3 ft) or more, such as bridge piers and foundations for bridges, buildings, and power plants.

These precautions include designing the concrete mix for the minimum practical strength; minimizing the cementitious material by using mineral admixtures in place of portland cement, lowering the water content (with lower slump or water-reducing admixtures) to maintain strength at a low cement content, and using the largest practical maximum size of coarse aggregate; and placing the concrete at a reduced temperature.

Grouts

Grouts can be defined as cementitious slurries that are injected into cracks, ducting, and other voids and fissures in concrete, or adjacent to concrete structures, to provide an impermeable barrier. The cementitious material used in a grout may be based on an organic or inorganic cement, but generally a portland cement is used when appreciable strength is also required. Grouts usually contain other materials besides cement and water. Sand may be used when a considerable volume of void space with a relatively open structure is to be filled. Mineral admixtures, such as fly ash and bentonite, are often used to help provide good fluid properties, without segregation occurring during injection. Fly ash provides additional cementitious action; bentonite is inert, but results in high permeability because it absorbs large amounts of water with a concomitant large increase in volume. Chemical admixtures are used to reduce the water content of a fluid slurry, to provide better cohesiveness, or to control setting times. Admixtures may also be added to counteract possible subsequent shrinkage of the grout. Aluminum powder is commonly used because the hydrogen generated by the alkaline corrosion causes expansion during setting [Eq. (20.3)]. A sulfoaluminate-based expansive component may also be used, which causes expansion during the first few days of curing. These admixtures ensure that the grout fully fills the available

[1] ACI Committee 116, "Cement and Concrete Terminology (ACI 116R-00)," American Concrete Institute, Farmington Hills, MI (2000).

space and counteracts subsequent shrinkage cracking. The important parameter of a grout is not initial or final set, but pumping time, which is a measure of the length of time the grout remains fluid enough to be properly injected. This is usually less than the time of initial set. Impermeability (or watertightness) is a more important consideration than strength. The grout's major purpose is to prevent entry of moisture or to prevent corrosion of steel, so that injection procedures should be such as to ensure proper filling of the total void space.

One application where grout plays an important structural role is in the construction of preplaced aggregate concrete. The void space in the aggregate is filled with a grouted mortar, which should be made with a *w/cm* ratio low enough to satisfy strength and durability requirements and should be air entrained if exposure conditions warrant it. Water-reducing admixtures and pozzolans help to attain the required flowability at the proper *w/cm* ratio; pozzolans also help to minimize bleeding and at the same time improve later strength and permeability. A large amount of grouting is done by drilling companies for oil and gas wells. Grouts are pumped down to form an impermeable barrier around well casings. Fortunately, the strength of such grouts need not be very high, but there are other quite critical requirements that have to be met. In deep wells, placement of the grout takes a considerable length of time and the grout must remain pumpable. Since the temperature of the wells can be quite high [generally over 200°C (390°F) and in the deepest wells over 400°C or 750°F], special cements have been developed that have long setting times (Chapter 3). Long pumping times increase the chance of segregation during injection, so special attention must be paid to the cohesiveness of the grout. Since the grout displaces the drilling mud used to cool the rig during the sinking of the well, mixing of grout and mud should be kept to a minimum. Generally, the sand used in the grout is finer than that used to make conventional concrete, with a fineness modulus in the range 1.3 to 2.3. Cement/sand ratios are usually in the range 1:1 to 1:2. The grout is mixed in special high-speed mixers, the sand being added to the cement–water slurry.

Refractory Concretes

By a suitable choice of materials, concretes can be made to perform satisfactorily at high service temperatures in furnace installations and as a substitute for refractory brick where awkward shapes are required or monolithic construction is desirable. Both cements and aggregates should be chosen for their refractory properties (Table 20.18). Calcium-aluminate cements should be used for refractory concretes. High-alumina cement is a rather impure mixture of calcium aluminate containing much iron and silica as well as CA. A much purer calcium-aluminate cement containing only calcium aluminates—CA, CA_2, and $C_{12}A_7$—should be used for very high temperatures. Aggregates should also be chosen for their refractory properties, using increasingly refractory materials as the service temperature rises. Reinforcement should be omitted or used sparingly in the form of mesh rather than bars, since increased elastic deformations and possibilities for corrosion can cause problems at high temperatures. Refractory concretes have a high compressive strength when cast due to the hydraulic bond. On heating, this bond breaks down in the range 800 to 1000°C (1470 to 1830°F) (1470 to), and the strength may fall to about 20% of the initial value. On further heating, the strength rises again, due to the formation of a ceramic bond.

TABLE 20.18 Types of Refractory Concrete

Type	*Service Temperature Limit (°C)*[a]	*Cement*	*Aggregate*
Structural	300	Portland	Common rocks
Heat resisting	1000	High alumina	Fine-grained basic igneous rocks (basalt, dolerite); heat-treated porous aggregates (bricks, scoria, expanded shale)
Ordinary refractory	1350	High alumina	Firebrick
Super-duty refractory	1450	Calcium aluminate	Firebrick
	1350–1600	High alumina	Refractory (magnesite, silicon carbide, bauxite, fired clay)
	—	Calcium aluminate	Refractory

[a] $°C \times 9/5 + 32 = °F$.

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PROBLEMS

20.1. What are the problems underlying the mix design of lightweight aggregate concretes?
20.2. Discuss the freeze–thaw resistance of concretes made with lightweight aggregates.
20.3. Why do lightweight concretes tend to show greater deformations in service than do normal-weight concretes?
20.4. Compare the properties of normal-weight and heavyweight concretes.
20.5. What special problems arise in the placement of heavyweight concretes?
20.6. Identify five different examples of architectural concrete in your community. Describe their appearance, and discuss how the architectural effects were obtained. Is there any apparent deterioration of the concrete or of its appearance?
20.7. What special materials that can be used for architectural effects are available at the building-supply stores in your locality?
20.8. Discuss the properties of (a) no-fines and (b) gap-graded concrete.
20.9. What are the advantages and disadvantages of no-slump concrete?
20.10. What are the special requirements of concretes used in massive structures?
20.11. Describe the production and use of grouts.

CHAPTER 21

Cement–Polymer Composites

Since concrete is a brittle material that is very weak in tension, it would be attractive to modify its behavior by an alliance with polymeric materials, which are ductile and relatively strong in tension. This has not been achieved in practice except for the use of polymeric fibers in fiber reinforced concrete. However, there is a new class of materials, Macro-Defect-Free Cements, which do indeed successfully marry the properties of the two materials and are described at the end of this chapter.

Currently, polymers are used primarily to enhance the durability of concrete. This can be done in four ways:

1. Protective coatings and sealants, which are applied on concrete surfaces, to prevent the ingress of moisture and deleterious chemicals.
2. Adhesives for bonding in repair and structural joints.
3. Impregnation of hardened concrete with a monomer followed by in situ polymerization (*polymer-impregnated concrete*).
4. Incorporation of polymer latexes into the concrete mix (*latex-modified concrete*).

In this chapter, we will only discuss latex-modified and polymer-impregnated concretes in which the polymers modify the properties of the bulk concrete rather than just the surface. Of course polymers also are used in small quantities ($< 1\%$ by mass of cement) as chemical admixtures, and these are discussed in Chapter 8. Polymers can also be used as the sole binder in polymer concretes. Asphaltic concretes and sulfur concrete come under this heading, but are clearly outside the scope of this book. Since polymers and monomers are much more expensive than traditional concrete materials, polymer-modified concretes are much more expensive. Relative material costs (conventional concrete = 1) are latex-modified concrete = 2–3; polymer-impregnated concrete = 3–6; polymer concrete = 8–20.

21.1 LATEX-MODIFIED CONCRETE

The most common method of combining concrete with a polymer is to add a polymer latex during mixing. The combination is called *latex-modified concrete* (LMC). A latex is an *emulsion*: a stabilized suspension of colloidal polymer beads [~0.1 mm (0.004 in.) dia.] in water.

Polymer Latexes

Most commercial latexes (Table 21.1) are based on thermoplastic (vinyl-type) polymers or elastomeric (rubber-like) polymers that readily form continuous coherent films when dried at room temperature. The beads are prepared by emulsion polymerization of the monomer. Most latexes have a solids content of about 45–55% (Table 21.2) with the emulsion stabilized with 6–7% of a water-soluble nonionic surfactant. The latexes are specially formulated to be compatible with the high-alkaline environment of concrete. Redispersible latexes of acrylates and vinyl acetates are now available that are free-flowing powders prepared by spray drying latexes. The powders have a coating of the surfactant and should redisperse to a stable latex on the addition of water. Redispersible latexes are not suitable for use in concrete expose to high humidity.

A schematic view of the film-forming process is given in Figure 21.1. Flocculation of the latex particles takes place as water is removed by evaporation. Coalescence of the particles to form a continuous film takes place as further moisture is removed by hydration. Thus, LMC is one exception to the general rule that extended moist curing is beneficial. In this case, after a brief period (1–2 days) of moist curing, the concrete should be allowed to dry out to promote film formation. The polymer film will form on the surfaces of capillary pores and at the paste–aggregate interface. It should therefore enhance the bond between aggregates and paste and limit the penetration of water through the capillary pore system (including ITZ pores). Because these films are relatively strong and tough in tension, they will increase the energy required to propagate microcracks in the matrix.

An alternative to the thermoplastic latexes is the use of water-dispersible thermosetting resins, such as epoxy. These are two-component polymer precursors (resin and hardener) that undergo final polymerization in the hardened concretes.

TABLE 21.1 Examples of Polymer Latexes Used in LMC

Polymer	*Type*	*Typical Applications*	*Wet Strength*
Vinyl acetate	Thermoplastic	Bonding agent	Low
Vinyl acetate-ethylene	Thermoplastic	Bonding agent	Moderate
Styrene-butadiene	Elastomeric	Overlays and patching	Moderate
Vinylidene chloride	Thermoplastic	Stucco	Good
Acrylic esters (acrylates)	Thermoplastic	Coatings and bonding agent	Moderate
Epoxy	Thermoset	Overlays	Good

TABLE 21.2 Typical Properties of Polymer Latexes Used in Concrete

	Vinyl Acetate	*Ethylene*	*Acrylate*	*Styrene Butadiene*
Solids Content	52%	55%	47%	47%
pH	5	5	9	10
Avg. Particle Size (μm)	2.0	0.8	0.2	0.2
Glass Transition Temp.	30°C	−3°C	12°C	5°C
Apparent Viscosity (MPa · sec)	1500	1500	150	90

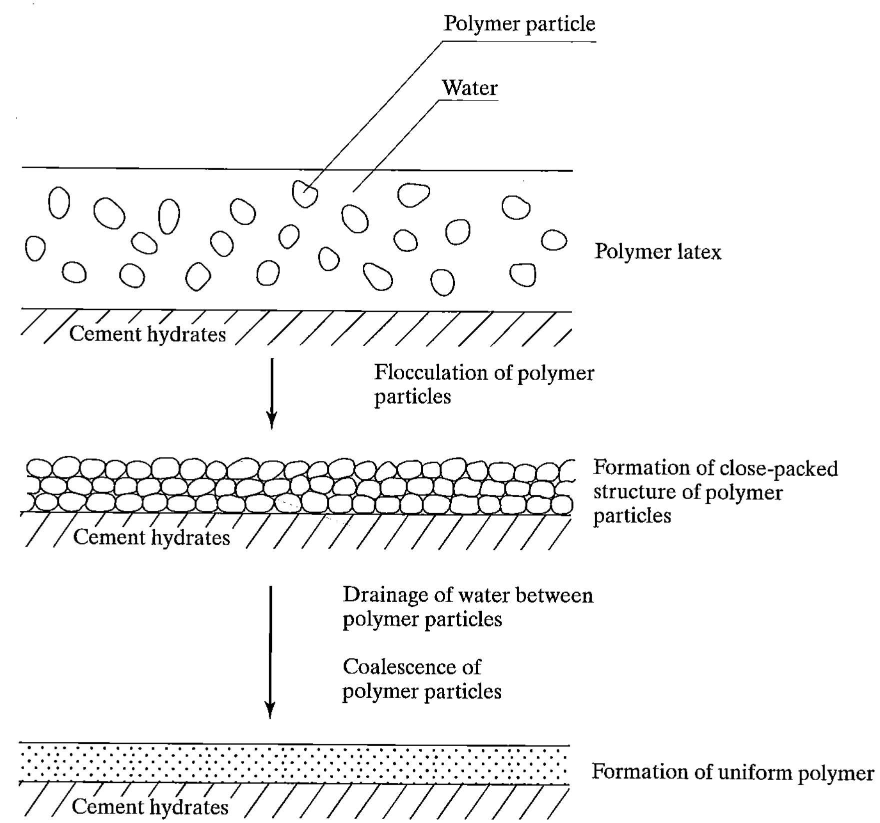


FIGURE 21.1
Schematic representation of polymer film formation in latex-modified concrete.

Effects on Properties

Fresh Concrete and Mix Design The addition of polymer latexes generally improves the properties of fresh concrete. The very small spherical particles of polymer act in a similar way to entrained air bubbles to improve workability and reduce bleeding. The surfactant also reduces water demand. Therefore, the *w/c* ratio can be reduced when latex is used. A good rule of thumb is to replace some of the mix water with an equal weight of latex. Typical values of the *w/c* ratio are 0.3 to 0.4.

Polymer films are expected to be chemically inert within the concrete. However, in some cases, hydrolysis in moist concrete due to the high pH of the local environment can cause the polymer film to break down. Poly(vinyl acetate) is particularly susceptible to hydrolysis, although its performance is improved when co-polymerized with ethylene or acrylic esters. Hydrolysis results in a severe loss of bond strength. More stable acrylate forms will slowly hydrolyze on prolonged exposure to moisture; co-polymerization with styrene improves its properties. Poly(vinyl chloride-vinylidene) co-polymers have been found to hydrolyze under field conditions, slowly releasing chloride ions into the concrete, which can cause corrosion of embedded metals.

A problem with the use of a latex may be a tendency to entrain excessive air due to the presence of the stabilizing surfactant. In some products, a defoamer is included; otherwise it may be necessary to add one. Generally, 15–20% solid latex by weight of cement is added. If the amount of latex is too high, a skin of latex may form at the surface. This skin can coalesce as water evaporates, making the surface very sticky and difficult to finish.

Latex-modified mortars are often used in patching, coatings, or floor-leveling applications, while latex-modified concretes are used in overlays. The sand contents used in LMC are generally higher (~45%) than those used in ordinary concretes (~40%).

Hardened Concrete Optimum properties are developed only when the dispersed polymer is converted to a continuous film within the concrete. Therefore, curing at 50% RH (or ambient RH) is recommended after initial moist curing. Once the polymer film has formed, it will remain stable up to about 95% RH.

Mechanical properties. The compressive strength of LMC is often higher than that of an unmodified concrete under dry curing conditions because strength continues to develop beyond 28 days. This is attributed not only to the development of the polymer film, but also to the fact that the polymer inhibits loss of water from LMC. Hence, the cement can hydrate more completely under these conditions. However, relative improvements in strength are greater under flexural and tensile loading (see Table 21.3). Also, LMC has a lower modulus of elasticity and higher strain at failure in tensile loading (Figure 21.2). The greater nonlinearity of the stress–strain curve indicates that more microcracking occurs prior to failure. It is believed that the polymer films inhibit the propagation of microcracks because of their high tensile strength. Furthermore, the ability of the film to form fibrils on rupture provides stress transfer behind the crack tip, thereby reducing stress concentrations and keeping cracks closed.

LMC also has improved bond both to old concrete and to reinforcing steel. In tests of bond with old concrete, failure generally occurs through the latter, not through

TABLE 21.3 Typical Mechanical Properties of Latex-Modified Mortars[a]

	Plain Concrete[b] 50% RH	*Plain Concrete*[b] 100% RH	*Styrene-Butadiene*	*VC–VC*[c]	*Acrylate*	*Vinyl Acetate*	*Epoxy*
Compressive strength							
MPa	30	40	35	60	32	25	50
lb/in.^{2}	4400	5800	5100	8700	4700	3600	7300
Tensile strength							
MPa	2	4	4.5	6.5	6	5	5
lb/in.^{2}	300	600	650	950	900	700	700
Flexural strength							
MPa	4	7.5	10	13	13	13	11
lb/in.^{2}	600	1100	1400	1900	1900	1900	1600
Modulus of elasticity							
GPa	25	–	11	16	–	–	18
10^6 lb/in.^{2}	3.6	–	1.6	2.3	–	–	2.6
Shear bond strength							
MPa	0.5–1.5	–	5[e]	>5[e]	>5[e]	>5[e]	–
lb/in.^{2}	70–200	–	>700[e]	>600[e]	>700[e]	>700[e]	–
Impact strength							
m · kg	0.07	0.08	0.22	–	0.25	0.18	–
in · lb	6	7	19	–	22	16	–
Abrasion resistance (% wear)	24	5	2.5	–	1.7	5	–

[a] Sand/cement = 3; polymer/cement = 0.20; dry-cured 28 days at 50% RH.
[b] Cured 28 days.
[c] Vinyl chloride–vinylidene chloride copolymer.
[d] Improved properties when co-polymerized with ethylene.
[e] Failure occurred through mortar rather than at the interface.

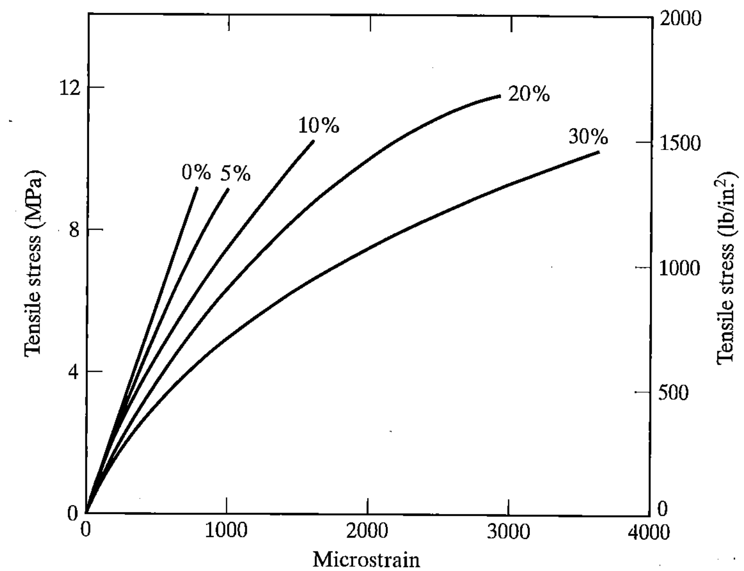


FIGURE 21.2

Stress–strain curves for latex-modified concretes containing polyvinyl acetate. (Polymer loadings in percent by weight of cement.) [From V. R. Riley and I. Razl, *Composites*, Vol. 5, No. 1, p. 28 (1974).]

the interface. Strengthening of the paste–aggregate bond along with a reduction in *w/c* ratio accounts for the increased mechanical strength.

Loss of mechanical strength will occur when LMC's are immersed in water (Table 21.4). Adsorption of water by the polymer film probably weakens the integrity of the polymer film. This decrease can be quite large if the polymer is degraded chemically by the high pH of the concrete pore solution. For this reason, most polymers will not perform well over long periods of immersion or contact with water. For example, vinyl chloride-vinylidene chloride co-polymer, which is very stable in the short term, gradually releases chloride ions under slow alkaline hydrolysis.

Volume Stability. The drying shrinkage and creep of LMC are lower than those of unmodified concrete. At least part of the reduction can be attributed to the use of lower *w/c* ratios, but the presence of the polymer film appears to increase the resistance of the matrix to applied and induced stresses.

Durability. The increased durability of LMC relative to plain concrete can be attributed to three factors. First, the polymer film lining capillary pore surfaces impedes water adsorption and permeability and prevents the entry of aggressive agents (Table 21.5). Second, the improved resistance to tensile cracking reduces the formation of a network of microcracks that can assist in water transport. When cracks do form, filaments of polymer, which bridge the cracks, keep them closed. Third, the lower *w/c* ratio provides generally improved durability. The air-entraining properties of the latex will naturally provide frost resistance. For these reasons, LMC has been used extensively in overlays on bridge decks and parking decks to prevent ingress of deicing salts.

TABLE 21.4 Typical Changes in the Strength of Latex-Modified Mortars[a] after Immersion in Water

	Strength in MPa (lb/in.^{2})							
	Compressive		*Tensile*		*Flexural*		*Shear Bond*	
	Dry[a]	*Wet*[b]	*Dry*	*Wet*	*Dry*	*Wet*	*Dry*	*Wet*
Control	17 (2500)	30 (4400)	1.5 (220)	2 (300)	4 (600)	5 (700)	0.3 (40)	1 (150)
Styrene-butadiene	35 (5100)	28 (4100)	4 (600)	2.5 (350)	10 (1500)	6.5 (950)	>5[d] (>700)	2.5 (350)
VC–VC[c]	60 (8700)	50 (7300)	–	–	9 (1300)	7.5 (1100)	>5[d] (>700)	4.5 (700)
Acrylate	40 (5800)	28 (4100)	6 (900)	3.5 (500)	13 (1900)	7 (200)	>5[d] (>700)	2.5 (350)
Vinyl acetate	25 (3600)	10 (1500)	5 (700)	0.5 (70)	13 (1900)	2 (300)	>5[d] (>700)	1 (150)
Epoxy	50 (7300)	48 (7000)	5.5 (800)	5 (700)	11.5 (1650)	11 (1600)	—	—

[a] Sand/cement = 3; polymer/cement = 0.2; cured 28 days at 50% RH.
[b] Immersed seven days after curing completed, before testing.
[c] Vinyl chloride–vinylidene chloride co-polymer.
[d] Failure occurs through substrate concrete.

TABLE 21.5 Chloride Ion Diffusion of Polymer-Modified Mortar and Concrete

		Diffusion Coefficient (m/s)	
Latex Addition	*Polymer : Cement Ratios*	*Mortar*	*Concrete*
None	–	6.4×10^{-10}	2.2×10^{-10}
Styrene-butadiene	0.10	6.4×10^{-10}	1.9×10^{-10}
	0.20	3.9×10^{-10}	0.9×10^{-10}
Acrylate	0.10	3.8×10^{-10}	0.6×10^{-10}
	0.20	2.4×10^{-10}	0.6×10^{-10}
Ethylene-vinyl acetate	0.10	4.4×10^{-10}	0.8×10^{-10}
	0.20	2.4×10^{-10}	1.0×10^{-10}

Applications

The high cost of LMC must be offset by improved performance. LMCs are ideal for repair work where high material costs are not so critical and where the development of a good bond with the existing substrate is a critical parameter. The best performing LMCs are ideal for thin overlays on bridge decks and parking decks to protect reinforcing steel from corrosion. Latexes based on elastomers and epoxies are most widely used for this purpose.

Acrylate latexes are used primarily to repair industrial floors that are subject to abrasion and contact with chemicals. Again, good bond strength is a key property. Acrylates are used extensively for spray coatings, either as stucco, as waterproofing for surfaces (e.g., basement walls), or in repair work. They have been used as finish coatings to cover and protect exterior insulation in residential buildings. LMCs are also used extensively for patching, for floor leveling, as a tile adhesive, and to obtain skid-resistant surfaces.

21.2 POLYMER-IMPREGNATED CONCRETE

When concrete is prepared and cured in the normal way, considerable capillary porosity is present in the cement paste, even in well-cured concretes. If the water present in the capillaries could be removed and replaced by a solid material, this would greatly improve the strength and durability of the concrete, by eliminating much of this void space. This is the basis of *PIC* (polymer-impregnated concrete). The problem of incorporating a solid into the pore system is solved by using a liquid *monomer* (the molecule that is the repeating unit of the polymer) to impregnate the concrete and subsequently polymerizing the monomer to form the solid polymer in situ within the pores.

Materials

Concrete Materials Any type of concrete can be successfully impregnated, regardless of the type of cement, admixtures, or aggregates used. Variations in mix design and materials will alter the amount of monomer needed for impregnation and the ease with which impregnation can occur. The amount of monomer needed for complete impregnation depends on the porosity of the concrete, which is determined by the *w/c* ratio, the amount of curing, the porosity of the aggregate, and so on. A more porous concrete (e.g., a concrete with a high *w/c* ratio) will require more monomer, but full impregnation will generally be achieved more easily and rapidly. The strength of the resulting PIC will be largely independent of the strength of the initial concrete, provided that full impregnation is achieved. Thus, the choice of initial materials can be determined largely by economic considerations.

This is not the case when partial impregnation is used, since the quality of the remaining unimpregnated concrete is obviously now of importance. The strength of PIC is still dependent on the final porosity of the material. Thus, if the pores of lightweight aggregate cannot be impregnated, lightweight PIC will not be very strong. Obviously, complete impregnation of porous, lightweight aggregate will require high polymer loadings.

Monomers and Polymers Many different monomers have been successfully used to produce PIC. The desirable properties of a monomer include low viscosity, relatively high boiling point, low toxicity, ease of polymerization, low cost, and availability. Methyl methacrylate (MMA), the monomer of plexiglass, and styrene are the most suitable of the common monomers. Their very low viscosities make them ideal liquids for penetrating the tortuous pore system of hardened concrete. Polyesters are too viscous to be successfully used alone, but when blended with styrene, their viscosities can be reduced sufficiently to be used for partial impregnation of concrete. The high volatility and toxicity of another common monomer, vinyl chloride, effectively rules out its use in PIC.

Process Technology

Production of PIC requires the following sequence of operations once the concrete has been cast and cured (Figure 21.3): (1) dry the concrete, (2) impregnate the concrete with monomer, and (3) polymerize the monomer.

Drying requirements For full impregnation of precast members, the removal of as much evaporable water as possible (2 to 4% by weight of concrete) is desirable if the optimum performance is to be achieved. A temperature of 150°C (300°F) is recommended for typical structural concrete. The time of drying depends on the drying temperature, the *w/c* ratio of the concrete, and the thickness of the specimen. At 150°C (300°F), a 100-mm (4-in.) thick concrete pipe will take about 24 h to dry. A rapid rate of drying may induce microcracking, but this is of little consequence since the cracks will also be impregnated with polymer. However, when only partial impregnation is desired, such as for the upper surface of a slab, cracking could be serious and

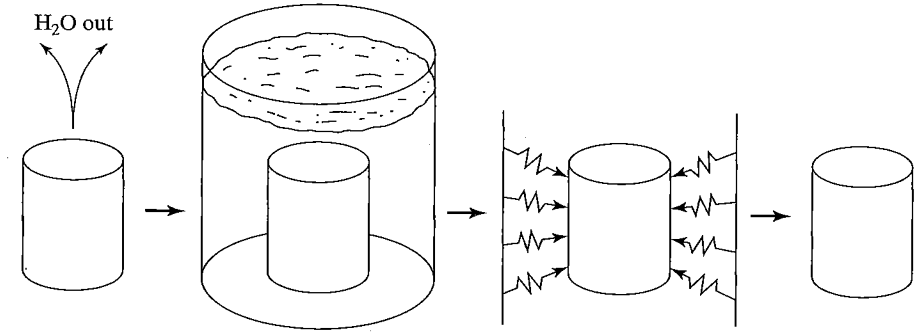


FIGURE 21.3

Process technology for polymer-impregnated concrete: (a) heating and drying; (b) impregnating with monomer; (c) polymerizing; (d) final product.

careful control of heating and cooling rates and the maximum drying temperature will be needed. Lower drying temperatures can be tolerated in many applications of partial impregnation. In situ temperatures of about 125°C (257°F) are achieved.

Impregnation Saturation of the concrete pore system depends on the viscosity of the monomer, the porosity and pore-size distribution of the concrete, the hydrostatic pressure, and time. Complete impregnation of good-quality dense concrete can be achieved after about 1 h of immersion if the dried concrete is first evacuated, the monomer introduced into the impregnation chamber under vacuum, and the monomer pressurized to about 70 kPa (10 lb/in.^{2}). This procedure can obviously only be used in a precasting plant.

Partial impregnation can be achieved satisfactorily by soaking at atmospheric pressure without prior evacuation. For example, 70 to 80% of total impregnation with MMA can be expected by soaking unevacuated specimens overnight. The depth of penetration will depend on soak time and the viscosity of the monomer. Depth of penetration is not linear with time and has been variously described as proportional to the logarithm of time or square root of time, thereby indicating a diffusion process. Low-viscosity monomers may penetrate the concrete faster, but tend to give less uniform impregnation. Bridge decks have been impregnated to a depth of 35 to 50 mm (1.4 to 2 in.) after soaking 8 to 12 h with MMA. The monomer is bonded on the surface and retained in a layer of sand to minimize losses from evaporation. Various methods of pressure impregnation for in situ impregnation are being developed.

Polymerization The ultimate success of the whole operation depends on proper polymerization. Monomers used in PIC can be polymerized in two ways: (1) by exposing them to gamma radiation (e.g., from a cobalt-60 source) or (2) by the use of a catalyst and heat (thermal-catalytic polymerization). Radiation-induced polymerization can take place at room temperature and occurs uniformly through relatively thick concrete sections, but the health hazards associated with the use of gamma radiation effectively

rule out its use in the field. Alternatively, a catalyst may be used to initiate polymerization. The catalyst–monomer mixture needs to be heated to 80 to 100°C (195 to 212°F) for polymerization to proceed. Heating breaks down the catalyst into free radicals, which are needed for polymerization. There are also chemicals called promoters that break down the catalyst at ambient temperatures. But thermal-catalytic methods are often preferred over the use of a promoter-catalytic system because polymerization will proceed faster at elevated temperatures and is more predictable. Steam, hot water, or infrared heating can readily be used as a heat source. When thermal-catalytic polymerization is used, care must be taken to minimize loss of monomer by evaporation. Wrapping the specimen in a plastic sheet or aluminum foil is convenient for small specimens; for large specimens, polymerization under hot water is most practical. The water provides the heat for curing and also prevents evaporation.

Properties of PIC

Mechanical Properties Table 21.6 shows that the compressive, flexural, and tensile strengths of a fully impregnated PIC are 2 to 4 times higher than those of the unimpregnated concrete. It should be noted, however, that the relationships between the different strengths remain unchanged and PIC behaves as a more brittle material. The stress–strain curve in compression remains linear up to about 75% of the ultimate load (Figure 21.4), and the departure from linearity is never very great. Consequently, failure under compression is sudden and the specimen shatters. Abrupt failure can be prevented by adding a monomer that gives rise to a tough, flexible co-polymer. Fracture mechanics studies indicate that fracture energies and the fracture toughness are higher for PICs than they are for plain concrete. Polymer impregnation strengthens the cement paste and improves the paste–aggregate bond. Fracture is therefore likely to occur through the aggregate, which may determine the strength limit of PIC. Weak, porous, lightweight aggregates are also strengthened by impregnation and still retain weight advantages over normal aggregates.

TABLE 21.6 Typical Properties of Plain Concrete and PIC

Properties	*Plain Concrete*	*PIC (Methyl Methacrylate)*	*PIC (Styrene)*
Compressive strength (28 days), MPa (lb/in.^{2})	40 (5800)	130 (19,000)	70 (10,000)
Tensile strength (28 days), MPa (lb/in.^{2})	3 (400)	11 (1600)	5 (900)
Flexural strength (28 days), MPa (lb/in.^{2})	5 (700)	18 (2600)	8 (1200)
Modulus of elasticity, GPa (10^6 lb/in.^{2})	25 (3.6)	45 (6.5)	50 (7.3)
Permeability of water, m/s (ft/yr)	5.0×10^{-13} (5.3×10^{-4})	1.3×10^{-13} (1.4×10^{-4})	1.4×10^{-13} (1.5×10^{-4})
Water absorption (%)	6.4	0.3	0.7
Coefficient of thermal expansion, 10^{-6}/°C (10^{-6}/°F)	8.0 (4.3)	9.5 (5.3)	9.0 (5.0)

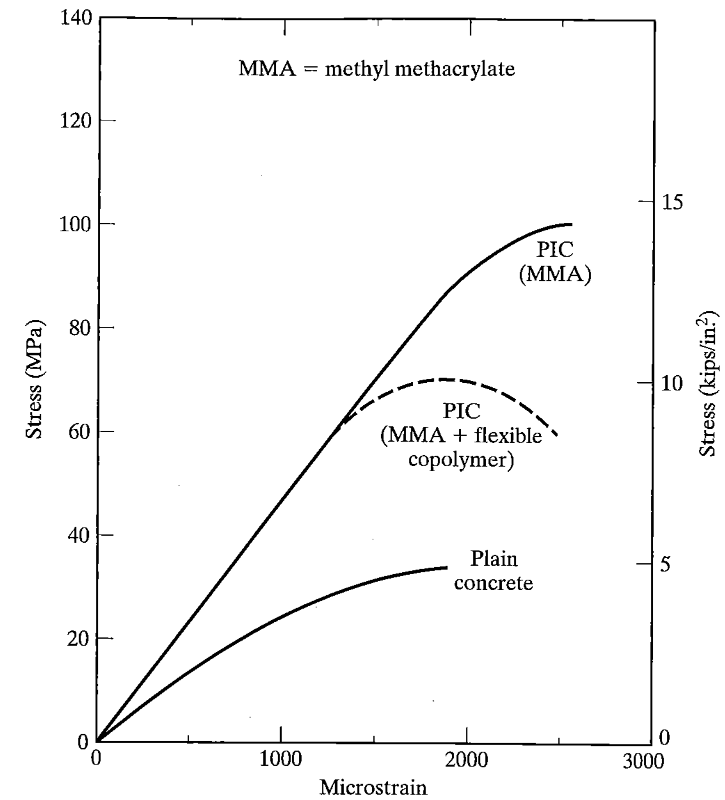


FIGURE 21.4
Stress–strain curve for PIC compared to plain concrete.

PIC still retains some porosity due to incomplete impregnation[1] or incomplete polymerization, and due to the fact that some shrinkage will occur during polymerization. Both the strength and the elastic modulus of PIC are dependent on its porosity. Radiation-cured PIC may show higher strengths than thermal-catalytically polymerized PIC due to more complete polymerization, particularly with monomers that polymerize less readily (e.g., styrene). The strength of PIC will fall off quite markedly with temperature, owing to the softening of the polymer. A cross-linking agent will improve the performance in this regard; PICs containing cross-linking polystyrene can be safely used up to 180°C (355°F).

Creep and Shrinkage Creep of PIC is reported to be very small, perhaps one-tenth of the creep of plain concrete. This is not surprising, because the concrete is strongly dried before impregnation; thus, no free water remains to cause creep of the cement paste. At higher temperatures, some increase in creep is observed as the polymer begins to soften. Similarly, no drying shrinkage would be anticipated, since the concrete

[1]The use of high pressure (up to 50 atm) can increase compressive strength by 25–30% due to more complete impregnation.

has already been dried. An important question is whether deleterious expansions will occur if water slowly penetrates the matrix, since this would obviously have an influence on long-term durability in moist environments. However, no problems of this kind have been reported in field trials.

Durability Many potential applications of PIC require only improved durability and do not need the large increases in mechanical properties that can be achieved. The great improvements in durability (see Table 21.7) that are obtained by impregnation are primarily the result of the marked decrease in permeability. Since the capillary pore system is now filled with polymer, aggressive chemicals can only attack the outer surface of the concrete and cannot penetrate deeply. Further, there is often a "skin" of polymer sealing the exterior of the concrete. Partial impregnation of concrete will give similar improvements in durability, since partial impregnation will effectively seal the surface layer of the concrete, even though improvements in mechanical properties are lower. When partial impregnation is used, care must be taken to ensure the development of a uniform impregnated layer.

Improvement in durability also is partially due to the fact that the concrete was dried before impregnation. For example, the improved freeze–thaw resistance would be expected, because there is no longer freezable water in the concrete. Since the entrained air voids will be mostly impregnated, air-entrained concrete need not be used. However, if partial impregnation is planned, air entrainment will be needed to protect the unimpregnated section. Although the durability measurements given in Table 21.7 look impressive, there are still no adequate long-term data available concerning exposure over several years. Field and laboratory tests will eventually provide this information, which is necessary for a proper evaluation of PIC.

TABLE 21.7 Durability of PIC[a]

	Plain Concrete	*PIC (MMA)*	*PIC (Styrene)*
Freeze–thaw			
Number of cycles	740	3650	5440
Weight loss (%)	25	2	2
Sulfate attack			
Expansion (%)	0.466	0.006	0.03
Number of days	480	720	690
Acid resistance (15% HCl)			
Weight loss (%)	27	9	12
Number of days	105	805	805
Abrasion resistance[b]			
Abrasion depth, mm (in.)	1.25 (0.050)	0.38 (0.015)	0.93 (0.037)
Weight loss (g) of specimens	14	4	6

[a]Based on data from "Polymers in Concrete: First Topical Report," Dept. BNL S0134, Brookhaven National Laboratory, (1968).
[b]Shot blast test; values per 1000 g of shot.

Fire Resistance Although polymers are flammable materials, they are not able to support combustion when distributed in PIC. Nevertheless, they can char and produce noxious fumes; fire retardants can be used to reduce these problems. More serious is the potential decrease in mechanical properties that can occur when PIC is exposed to higher temperatures. This will be a severe limitation on the use of PIC in high-strength applications.

Applications

Most applications that have been suggested for PIC take advantage of the tremendous improvements in durability. Since the production of PIC is expensive, because of the high cost of the monomer compounds and the impregnation process, the high cost of PIC must be justified in terms of a long maintenance-free service life or for applications where replacement would be difficult and costly.

Full impregnation can only be used on precast products. Applications that have been the subject of testing programs include concrete sewer pipe, precast segments for tunnel linings, underground support systems for mining operations, railroad ties, and precast pilings for wharves and jetties. Partial impregnation has advantages in its greater potential for field use. There has been a great deal of interest in the use of polymer impregnation to provide an effective barrier against deicing salts. Techniques for field impregnation are being explored, and this promises to be the focus of considerable inventiveness and ingenuity. A precast PIC bridge deck system has also been proposed. Impregnation of floor slabs used in corrosive industrial environments is another possibility.

The use of PIC is still relatively limited, because of the high cost of production and the availability of more cost-effective alternatives with comparable performance. The use of a pure polymer binder, obtained by mixing aggregates with liquid monomer and then polymerizing, can give comparable properties. Although the material costs are higher for polymer concrete, due to much higher monomer contents, processing costs are much lower.

21.3 ORGANOCERAMICS

The most recent development in the field of cement–polymer composites is the so-called *Macro-Defect-Free (MDF) Cements* developed in the early 1980s. These materials are formed by roll-milling techniques used in the plastics industries. The name was adopted because the polymer was considered to be only a processing aid to help eliminate the "macro-defects" in cast cements that reduce tensile strength to low levels. It is now known that the material actually involves chemical and physical synergy between the cement and polymer to help develop an intricate composite structure with ceramic-like properties. In contrast, LMC and PIC only modify the conventional concrete structure. For this reason, the generic name *organoceramic* (or organocement) is to be preferred.

TABLE 21.8 Formulation of Organoceramics (weight %)

		CAC-PVA	CAC-PFR
Cement:	Calcium Aluminate Cement	84.3	79.8
Polymer:	Poly(vinyl alcohol)	5.9	–
	Phenol-formaldehyde resin	–	10.4
	Nylon polymer	–	1.4
Plasticize:	Glycerin	0.6	1.8
Solvent:	Water	9.3	–
	Methanol	–	8.2

Materials and Processes

Calcium aluminate cement gives the best properties because the reactions with the polymer are optimal. Either water-based or methanol-based formulations can be used (Table 21.8). The ingredients are combined in a low shear planetary mixer and then roll-milled at a shear rate of >1000 sec^{-1} for 1–2 min. During this time, the cement particles are close-packed and release ions from hydration that promote light cross linking of the polymer. This increases viscosity, giving the desired viscoelastic properties required in the final forming step. The material is usually calendared into sheets, which are then shaped or trimmed to size, but it can also be extruded. The material is then heated to remove excess solvent and complete cross linking of the polymer.

The material forms an unusual microstructure. The bulk of the cement remains unhydrated and the particles act as a microfiller. In fact, up to 65% of the cement can be replaced by other inert fillers of similar size. Thus, the composite is really a micromortar. The binder has two components: a cross-linked polymer phase and an interphase (Figure 21.5). The latter is a nanocomposite of polymer and hydration products. In the poly(vinyl alcohol) system, the interphase is a continuous second phase interpenetrating the polymer matrix, giving it extremely high strengths.

Properties and Applications

The principle feature of these composites is their very high flexural strengths and fracture toughness, as shown in Table 21.9. This is a clear indication that the polymer is contributing effectively to the behavior of the composite. Composites made with water-soluble polymers are rather sensitive to moisture, since the polymer matrix absorbs water and becomes soft. The cross links are eventually broken and the cement starts to hydrate again. However, the material is quite stable at high humidities (<95% RH), and surface treatments have been used to protect direct contact with water. These materials are particularly suited for lightweight components of thin sections and have been used as interior cladding in buildings. However, currently there are few commercial applications since the material is still new and requires quite different processing from that used in normal construction.

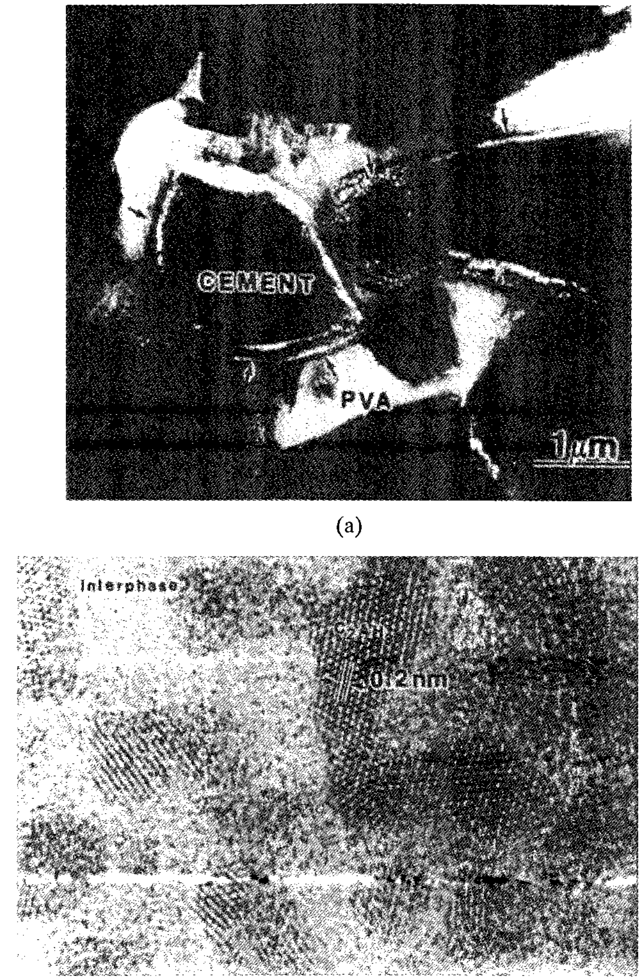


FIGURE 21.5

Microstructure of a CAC-PVA organoceramic by transmission electron microscopy. (a) Bright field image (general view); interphase is marked by arrows. (b) High resolution image of interphase showing nanocrystals of hydrate in an amorphous matrix.

TABLE 21.9 Properties of Organoceramic

Property	CAC-PVA	CAC-PFR	Cement Paste
Compressive strength, MPa	500	300	70
Flexural strength, MPa	250	190	20
Modulus of elasticity, GPa	45	45	45
Fracture toughness, $MPa/m^{1/2}$	3	3	0.3
Density kg/m^3	2500	2200	2700
Thermal expansion, 10^{-6}/C	10	18	15
Water absorption % (90 days)	4.0	0.7	–
Wet flexural strength, MPa (90 days immersion)	50	190	20

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PROBLEMS

21.1. Describe the differences between latex-modified concrete and polymer-impregnated concrete (a) in process technology; (b) in concrete properties.

21.2. How do different monomers affect the performance of PIC?

21.3. How does the initial porosity of concrete affect the properties of PIC?

21.4. What advantages would (a) latex-modified concrete and (b) polymer-impregnated concrete have in bridge construction?

21.5. What is the effect of moisture, during and after curing, on the properties of latex-modified concrete?

C H A P T E R 22

Fiber Reinforced Concrete

As we have already seen (Chapter 13), plain concrete is a brittle material, with low tensile strength and strain capacities. To help overcome these disadvantages, there has been a steady increase, since the late 1960s, in the use of *fiber reinforced concrete* (FRC). Fiber reinforced concrete may be defined as a concrete incorporating relatively short, discrete, discontinuous fibers. Generally, the fibers are not added to increase the concrete strength, though modest increases in strength may occur. Instead, the principal role of the fibers is to control the cracking of the FRC and then to modify the behavior of the material once the concrete matrix has cracked; by *bridging* across the cracks as they begin to open, fibers provide *post-cracking ductility* to the FRC.

The use of FRC has been increasing steadily in recent years. As of 2001, over 76 million m^3 (100 million yd^3) of FRC were produced annually, with the principal applications being slabs on grade (60%), shotcrete (25%), and precast members (5%), and the remainder of the production distributed among a number of other specialty structural forms. A number of different types of fibers are used to produce FRCs of various kinds. The most common ones are steel, organic polymers (primarily polypropylene), glass, carbon, asbestos, and cellulose. These fibers vary considerably in geometry, properties, effectiveness, and cost. Typical properties of the most common fiber types are given in Table 22.1. A brief description of some of these fibers follows.

Steel fibers may be produced either by cutting wire, by shearing sheets, or from a hot-melt extract. The first generation of steel fibers were smooth, but it was soon found that, as a result, they did not develop sufficient bond with the cementitious matrix; modern steel fibers are generally either deformed along their lengths or at their ends (Figure 22.1) to enhance the cement–fiber bond. Though they will rust visibly when exposed at the concrete surface, they appear to be highly durable within the concrete mass. In some cases, such as high-temperature refractory applications, stainless steel fibers may be required. Some of the properties of steel fibers are specified in ASTM A 820.

Glass fibers are produced in a process in which molten glass is drawn, in the form of filaments, through the bottom of a heated platinum tank or bushing. Most commonly, 204 filaments are drawn simultaneously; after solidification, they are collected into a

TABLE 22.1 Typical Properties of Fibers and Cement Matrix

Fiber	*Diameter* (μm)	*Specific Gravity*	*Modulus of Elasticity* (*GPa*)	*Tensile Strength* (*GPa*)	*Elongation at Break* (%)
Steel	5–500	7.84	200	0.5–2.0	0.5–3.5
Glass	9–15	2.60	70–80	2–4	2.0–3.5
Asbestos					
Crocidolite	0.02–0.4	3.4	196	3.5	2.0–3.0
Chrysotile	0.02–0.4	2.6	164	3.1	2.0–3.0
Polypropylene (monofilament or fibrillated)	6–200	0.91	5–77	0.15–0.75	15
Aramid (Kevlar)	10	1.45	65–133	3.6	2.1–4.0
Carbon					
PAN*	7–9	1.6–1.7	230–380	2.5–4.0	0.5–1.5
Pitch	9–18	1.6–2.15	28–480	0.5–3.0	0.5–2.4
Nylon	20–200	1.1	4.0	0.9	13–15
Cellulose	–	1.2	10	0.3–0.5	–
Polyethylene	25–1000	0.95	0.3	0.08–0.6	3–80
Sisal	10–50	1.5	13–26	0.3–0.6	3–5
Wood fiber (kraft pulp)	25–75	1.5	71	0.7–0.9	–
Cement matrix (for comparison)	–	2.5	10–45	0.004	0.02

*Polyacrilonitrile based.

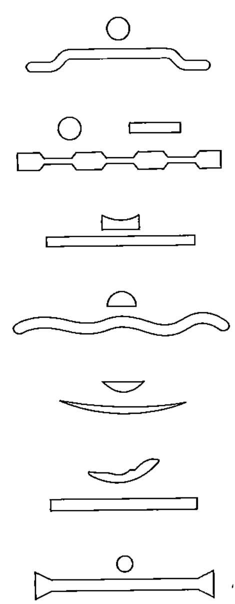


FIGURE 22.1

Typical cross sections and profiles of commonly available steel fibers.

strand consisting of the 204 filaments. Glass fibers are available both as "chopped strand" and as a continuous roving. Ordinary borosilicate glass fibers (E-glass) and soda-lime glass fibers (A-glass) are not suitable for use in concrete, since attack by the highly alkaline environment in concrete will rapidly reduce the strength of the fibers. This has led to the development of alkali resistant (AR) glass fibers, containing from about 16% to 20% of zirconia (ZrO_2). Such glass fibers are used primarily in thin sheet components, such as architectural panels.

Naturally occurring *asbestos fibers* have been used with cement and water in the manufacture of asbestos cement pipe and other building components since about 1900. Asbestos fibers are particularly compatible with the cement matrix, which makes it easy to disperse relatively large volumes of fibers in the matrix and enhances the fiber–matrix bond in the hardened composite. The resulting asbestos cements have high corrosion and abrasion resistance. However, there are significant health hazards associated with the production and handling of asbestos fibers, leading to lung cancer. (It should be noted that health risks are minimal during the *use* of asbestos cement components, as long as they do not begin to disintegrate due to mechanical abuse). Thus, as a result of the health risks and increasingly strict environmental regulations, asbestos fibers have largely been replaced with other types of fibers (primarily cellulose).

Synthetic (polymeric) fibers have become increasingly common in recent years. Most synthetic fibers have lower elastic moduli than concrete. At the relatively low fiber volumes currently used in industrial practice (<0.5%), they are most effective in reducing the amount of plastic shrinkage cracking, though they also provide some toughening and impact resistance. *Polypropylene fibers* are by far the most common low-modulus synthetic fibers, although *nylon* and *polyethylene* fibers are also in use. *Carbon fibers* and *Aramid (aromatic polyamide) fibers* are high-modulus synthetic fibers that also have high tensile strengths. They are very effective in FRC, but their high cost has limited their use.

A number of *natural organic fibers* have also been used in FRC, such as sisal, jute, coir, elephant grass, and sugar cane bagasse. These are all low-modulus fibers that tend to deteriorate in damp or alkaline environments unless specially treated. They have been used primarily in the production of low-cost housing elements in developing areas of the world. *Cellulose (wood pulp) fibers*, however, which have a higher elastic modulus and tensile strength than other natural fibers, have been used extensively as a replacement for asbestos fibers. They do, however, need special processing before they can be used in FRC.

It must be emphasized that, in general, *fiber reinforcement is not a substitute for conventional steel reinforcement.* Fibers and steel reinforcement play different roles in concrete: Reinforcing bars are used to increase the load-bearing capacity of structural concrete members, while fibers are more effective for crack control. There are many applications in which fibers can be used effectively in conjunction with conventional reinforcement to improve the behavior of structural components, for instance when the concrete is to be subjected to blast or impact loading, or in seismic applications.

22.1 DEFINITIONS

Before discussing the mechanics of fiber reinforcement, it is first necessary to define several parameters:

1. *Aspect ratio* = (fiber length/equivalent fiber diameter), where the equivalent diameter is the diameter of a circle having the same cross-sectional area as the fiber. For reasons of workability and dispersion in the matrix, the aspect ratio of most modern fibers is in the range of 50 to 150.
2. *Critical length*, l_c = length above which the fiber will fracture rather than pull out when a crack intersects the fiber at its midpoint. For frictional shear stress transfer along a circular fiber, the calculated value of l_c is

$$l_c = (\sigma_{fu} r)/\tau_{fu} \qquad (22.1)$$

   where σ_{fu} is the ultimate fiber strength, r is the fiber radius, and τ_{fu} is the maximum frictional shear stress. The effects of fiber length on shear stress transfer and l_c are shown in Figure 22.2.
3. *Orientation factor, or fiber efficiency factor* = efficiency with which randomly oriented fibers can carry a tensile force in any one direction. Depending upon the particular assumptions made, this factor can range from about 0.2 to 1.0.
4. *Spacing factor*. If the fibers are close enough together, the first cracking strength (see subsequent discussion) of the composite is higher than that of the matrix

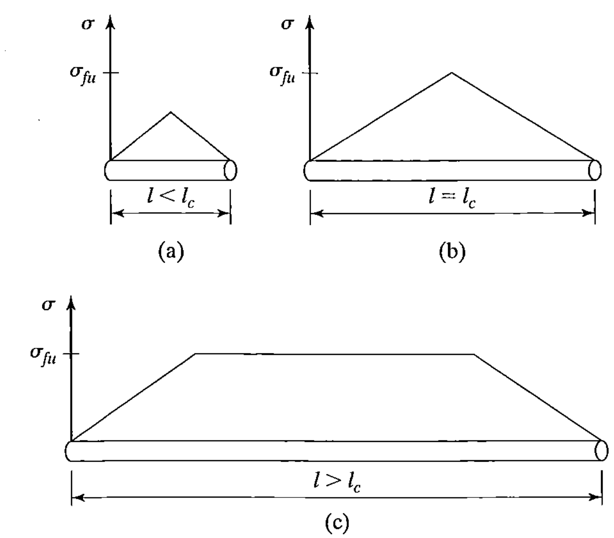


FIGURE 22.2

Effect of fiber length on shear stress transfer in a cement matrix.

alone, because the fibers effectively reduce the stress intensity factor at the crack tip, which controls fracture. A typical expression for the average fiber–fiber spacing for cylindrical fibers is

$$S = (Kd)/V_f^{1/2} \qquad (22.2)$$

where S is the fiber spacing, K is a constant, d is the fiber diameter, and V_f is the fiber volume content.

5. *First crack strength* = the stress corresponding to the load at which the load versus deflection curve (in tension or flexure) of the FRC first exhibits a significant nonlinearity.

22.2 FIBER–MATRIX BOND

For a composite system such as FRC, the mechanical behavior depends not only on the properties of the fibers and the cementitious systems, but also on the bonding between them. The nature of the fiber–cement interface is particularly complicated, since there may be chemical reactions between the cement and some types of fibers. Also, the nature of the interface may change with time as the cement matures or undergoes time-dependent volume changes. Generally, the particulate nature of the fresh concrete mix leads to the formation of water-filled spaces around the fibers, due to

1. bleeding of water around the fibers and
2. inefficient packing of the ~10 μm (0.4×10^{-3} in.) cement grains in the zone out to about 50 μm (2.0×10^{-3} in.) from the fiber surface.

Hence, close to the fiber surface, the matrix is more porous than it is in the "bulk" cement paste. This is shown schematically in Figure 22.3 and is typical for monofilament fibers. For "bundled" fibers, where the reinforcing unit consists of a bundle of closely spaced filaments (as in glass fiber reinforced concretes), the cement grains may be unable to penetrate into the spaces between the individual filaments. In such materials, the outer filaments may be well bonded to the matrix, but the inner filaments will not be.

For properly designed FRC mixtures, the primary mode of failure is by fiber pull-out, since this consumes much more energy than is involved in breaking the fibers and

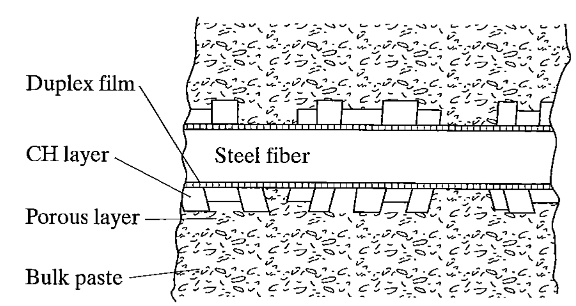


FIGURE 22.3

Schematic description of the interfacial transition zone in steel fiber-reinforced concrete.

leads to much better utilization of the fibers. The general form of the bond is fairly well known for the different types of fibers. For *steel* fibers, there is a combination of adhesion, friction, and mechanical interlock. For *glass* fibers, there is in addition some chemical reaction between the cement and the glass; in particular, alkali attack tends to weaken the fiber reinforcement, though to a much lesser extent with the AR glasses. With the *organic* fibers, the bond is primarily due to mechanical interlock.

It is now common practice to increase the fiber–matrix bond strength, by deforming the fibers along their lengths or at the ends (Figure 22.1). Large increases in bond strength are not reflected in similar increases in the strength of the FRC, but will improve the post-cracking behavior. Table 22.2 shows typical pullout strengths for a number of different fibers in various matrices.

22.3 MECHANICS OF FIBER REINFORCEMENT

As stated earlier, the principal role of the fibers is to bridge across the cracks that develop in the matrix (i.e., in the *post-cracking zone*), when the strain of the composite has exceeded the ultimate strain capacity of the brittle matrix. A typical stress–strain curve for low fiber volume (<1%) FRC is shown in Figure 22.4. Point A represents the load at which the matrix begins to crack (first-crack strength). Usually, this is at about the same stress at which cracking occurs in plain concrete, and thus the segment OA is essentially the same for both plain and fiber reinforced concrete. Thus, there is little fiber influence on the strength of the FRC, but the post-peak toughness and load-bearing capacity are greatly increased. In the post-cracking zone, the fibers may increase the strength of the FRC over that of the matrix by transferring loads across the cracks. They increase the toughness by providing energy absorption mechanisms through the gradual debonding and pull out of the fibers bridging the cracks. Figure 22.5 shows a schematic representation of the stress field around an advancing crack in FRC. There is a traction-free zone, where the crack is wide enough for all of the fibers to have pulled out; a fiber-bridging zone, in which stresses are transferred by frictional slip of the fibers; and a microcracked matrix process zone, with enough aggregate interlock to transfer some stress within the matrix itself. It should be noted that in the first part of the descending branch of the stress–strain curve, the fiber stress is substantially below the yield stress, so that yielding of the fibers does not occur. However, if the fibers are long enough to maintain their bond, they may eventually fail by yielding or fracturing at high strains, near the tail of the curve.

TABLE 22.2 Typical Fiber-Matrix Pullout Strengths

Matrix	*Fiber*	*Pullout Strength MPa*	*(lb/in.²)*
Cement paste	Asbestos	0.8–3.2	(115–460)
	Glass	6.4–10.0	(930–1450)
	Polycrystalline alumina	5.6–13.6	(810–1970)
	Steel	6.8–8.3	(990–1200)
Mortar	Steel	5.4	(780)
Concrete	Steel	3.6	(520)(first crack)
		4.2	(610) (failure)
	Nylon	0.14	(20)
	Polypropylene	1.0	(150)

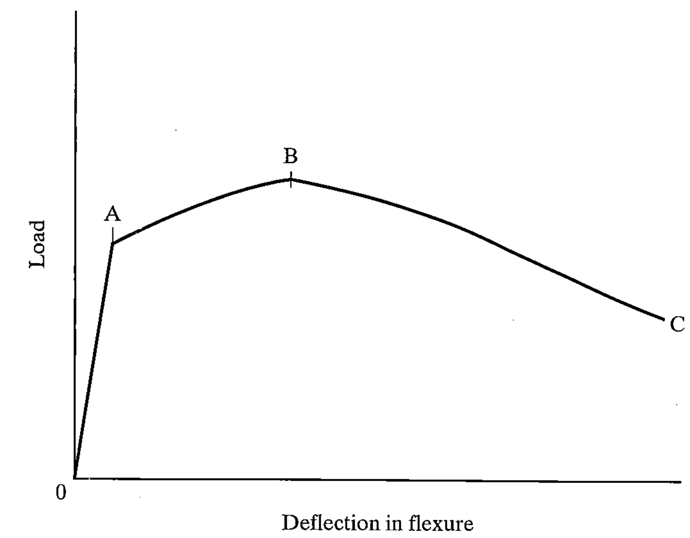


FIGURE 22.4
Typical load–deflection curve for fiber reinforced concrete in flexure.

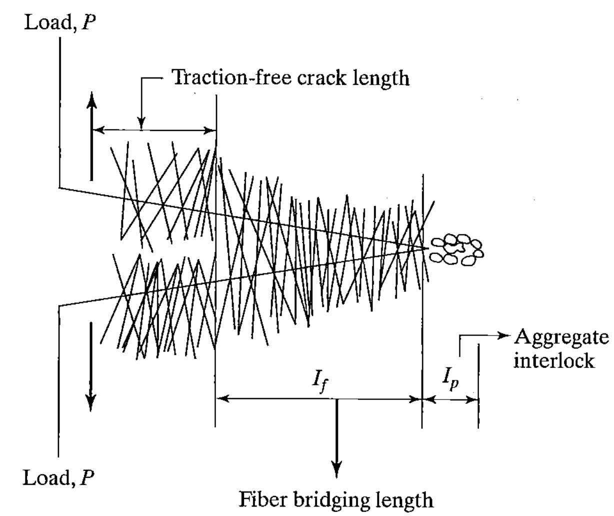


FIGURE 22.5
Schematic representation of fibers bridging across a crack.

22.4 FABRICATION OF FRC

Mix Proportioning

For the fiber volumes currently used in practice (<1.0% for steel, <0.5% for polypropylene), the procedures for mix proportioning are essentially the same as those used for plain concrete, since the concrete properties are controlled largely by the *w/c* ratio and the cement content. However, the addition of fibers will reduce the workability of the concrete. This can be generally be compensated for by increasing the fine-to-coarse aggregate ratio and by increasing the cementitious material content, generally through the addition of pozzolanic materials. As with plain concrete, this is something of a trial and error method; trial mixes must be used to adjust the proportions of the constituent materials.

It should be noted that the standard slump test has been found to be an inappropriate measure of the workability of FRC, since FRC mixtures with apparently low slump values can be quite workable in the field. For proper compaction, FRC mixtures must be placed using vibration, and so workability tests that involve vibration are more appropriate. It is therefore recommended that either the *Vebe test* (Chapter 9) or the *inverted slump cone test* (ASTM C 995) be used. The latter test, developed specifically for FRC, involves the determination of the time required for the FRC to flow through an inverted standard slump cone under internal vibration.

Apart from possible difficulties with workability, it is also more difficult to compact FRC; in particular, too high a coarse aggregate content can greatly decrease the compactibility. While for most fiber mixes external vibration is preferred, this may not be practical in the field. In general, fibers tend also to reduce the bleeding and improve the cohesion of a concrete mix.

Fabrication of Fiber Reinforced Concrete

FRC can in general be produced using the same equipment and procedures as for plain concrete. The basic problem is to ensure that the fibers are *uniformly dispersed* throughout the concrete, by avoiding segregation or balling of the fibers. The problem becomes more severe as the aspect ratio, the fiber volume, or the maximum size of coarse aggregate increase. Steel fibers are prone to *clumping* (balling). They should always be added to the mix in a clump-free state. For transit mix concrete, they should generally be added last to the wet concrete, and 30 to 40 revolutions of the drum at mixing speed should properly disperse them. Alternatively, the fibers could be added to the aggregates on a conveyor belt during the addition of the aggregates to the mix. More commonly now, *collated* steel fibers are used, which are held together by a water-soluble sizing that dissolves during the mixing process, thereby eliminating the clumping problem. It is difficult to add more than about 0.2% of fibrillated polypropylene fibers to a normal concrete mix, because of workability problems. However, monofilament synthetic fibers can be added relatively easily up to at least 0.5% by volume.

FRC can be placed in much the same way as plain concrete; while it may appear to be very stiff, if the mix is properly designed, it will flow readily under vibration. It can also be pumped successfully. There have more recently been very extensive applications of fiber reinforced *shotcrete* (see Chapter 11), primarily for tunnel linings and for surface repairs. The advantages of this technique, particularly in tunnel linings, are that the fiber shotcrete can be placed robotically (for reasons of worker safety in mining applications), and without the necessity of first fastening a steel mesh to the substrate as is commonly done with plain shotcrete. For high-volume (>3%) fiber applications, *extrusion* techniques are used that can result in composites combining both very high strength and greatly enhanced toughness.

For glass fibers, a "spray-up" method is generally used to produce thin sheets. Using a special pump and spray gun, the fibers are chopped and combined with a cement slurry and then sprayed onto a mold. This method can also be used with carbon and other synthetic fibers. Up to 10% by volume of fibers can be introduced using this technique. To obtain a more efficient fiber orientation, the "winding" process may be used with continuous fibers or filaments. The fibers are passed through a cement slurry, then wound on a frame; additional slurry and chopped fibers may then be sprayed on to achieve the desired thickness. With this method, a fiber volume of up to 15% can be achieved.

To produce FRCs with very high fiber contents (>5% by volume), two further techniques should be mentioned. *SIFCON* (slurry infiltrated fiber concrete) is prepared by placing the fibers in the forms, and then infiltrating them with a fine-grained mortar slurry. *SIMCON* (slurry infiltrated mat concrete) is produced by infiltrating steel fiber mats with a cementitious slurry. In both cases, very high flexural strength and toughness may be achieved; the major disadvantage is the extremely high cost of the fibers themselves. However, for specialty repairs, or to withstand explosive loading, these materials offer great possibilities.

22.5 PROPERTIES OF FIBER REINFORCED CONCRETE

Strength

It cannot be emphasized too strongly that, at the fiber volumes used in normal commercial applications, *the role of the fibers is not to increase strength*, though modest strength increases may occur. This is shown clearly in Figure 22.6 for compression and in Figure 22.7 for flexure. Even in direct tension, where it would be expected that fibers would be most effective in terms of strength, the data in Figure 22.8 indicate only about a 30% increase in strength for a steel fiber volume of 1.5%. If what is desired is a strength increase, it is clearly much easier (and much cheaper) simply to redesign the plain concrete mix, primarily by reducing the *w/c* ratio.

Similarly, fibers have no significant effects on either the shear strength or the torsional strength of concrete. They also have no particular effect on the elastic modulus.

Toughness

The principal role of the fibers is to bridge across the cracks that develop in concrete as it is loaded (or as it dries). If the fibers have sufficient strength and stiffness, and if they can achieve sufficient bond with the matrix, they will tend to keep the crack widths small and will permit the FRC to withstand significant stresses over a relatively large strain capacity in the post-cracking (or *strain-softening*) stage. Thus, the fibers can provide a considerable amount of post-cracking "ductility." In other words, they increase

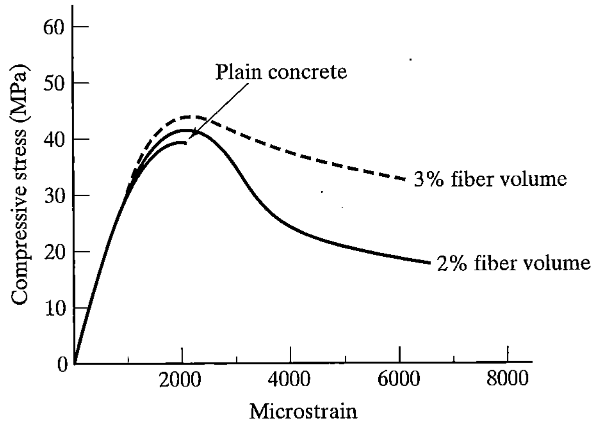


FIGURE 22.6

Stress–strain deformation in compression of steel-fiber concrete. [From report by RILEM Technical Committee 19-FRL, *Materials and Structures (Paris)*, Vol. 10, No. 56, pp. 103–120 (1977).]

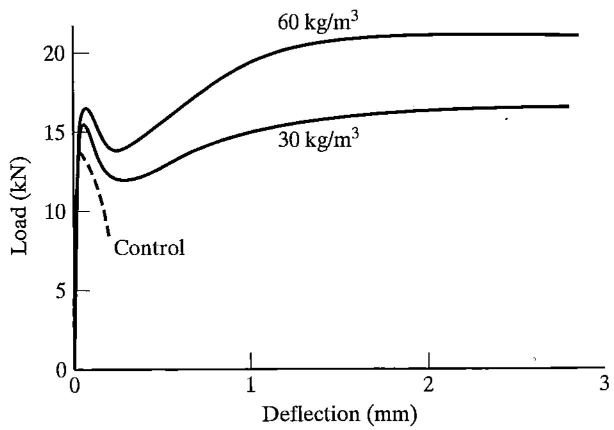


FIGURE 22.7

Influence of fiber content on load-deflection curves: 50-mm (1.97-in.) long hooked-end fibers. [Adapted from P. N. Balaguru and S. P. Shah, *Fiber-Reinforced Cement Composites*, McGraw-Hill (1992).]

the *toughness* of the FRC. Here, toughness will be defined as the area under the load-deflection curve obtained from a static test of a specimen up to a specified deflection; it is a measure of the energy absorption capacity of the FRC.

As may be seen from Figures 22.6, 22.7, and 22.8, while fiber additions do not significantly increase concrete strength, they have an enormous effect on toughness (i.e., on the area under the load-deflection curves under all types of loading). The increase in toughness occurs almost entirely in the post-cracking (descending) parts of the curves. Any increase in fiber volume, regardless of the fiber type, increases the toughness. However, some fibers are more effective than others in this regard. For instance, deformed fibers, which have a better bond with the matrix, will be more effective than smooth fibers of the same material, which pull out of the matrix more easily. As well, from Figure 22.9, it may be seen that steel fibers (because of their higher stiffness) are more effective than polypropylene fibers in this regard.

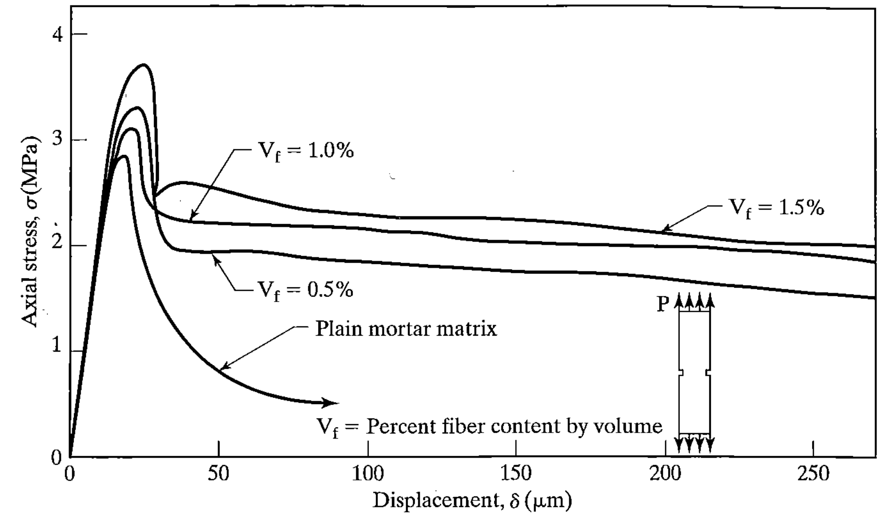


FIGURE 22.8

Stress–displacement curves obtained from direct tension tests on plain mortar matrix and SFRC. [Adapted from ACI Committee 544, *Fiber Reinforced Concrete*, ACI 544.1R-96, American Concrete Institute, Farmington Hills, MI (1996).]

Schematic	Fiber	Material	Geometry	Length, mm	Cross section	Tensile strength, MPa	Elastic modulus, GPa
(straight line)	F1	Polyolefin	Undeformed	50	Circular	365	2.6
(crimped)	F2	Polypropylene	Sinusoidal crimping	30	Circular	450	3.5
(crimped)	F3	Polypropylene	Sinusoidal crimping	50	Circular	450	3.5
(flat-ended)	F4	Steel	Flat-ended	30	Circular	1198	200

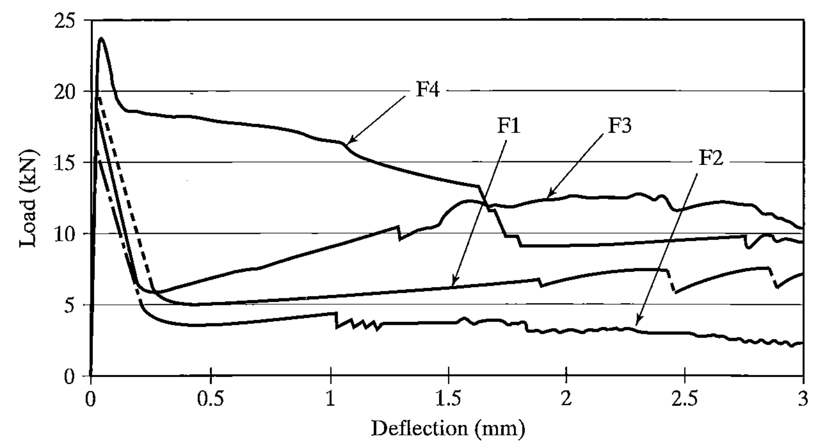


FIGURE 22.9

Flexural response of FRCs made with various fibers under static loading. [From V. Bindiganavile and N. Banthia, "Polymer and Steel Fiber-Reinforced Cementitious Composites under Impact Loading—Part I: Bond-Slip Response," *ACI Materials Journal*, Vol. 98, No. 1, pp. 10 – 16 (2001).]

It must be remembered that it requires much more energy to pull a fiber out of the matrix than to simply break the fiber. Thus, to use the fibers most effectively, they should be designed to fail by pulling out of the matrix at loads as close as possible to those required to break the fibers. This is a function both of fiber type and fiber geometry (length, surface deformation, fiber profile). There is a great deal of work currently underway to try to optimize fiber properties, so that FRC can be "tailor-made" for any particular application.

Toughness is generally measured under flexural loading, since it is this type of loading that FRC most commonly is required to withstand. The two most common methods for determining flexural toughness and residual strength are those prescribed in ASTM C 1018 (Figure 22.10a) and in the Japanese standard JSCE SF-4 (Figure 22.10b). In both tests, either molded or sawed specimens [typically of dimensions 100 × 100 × 350 mm (4 × 4 × 14 in.)] are loaded slowly in four-point bending (third-point loading), and the load versus center-point deflection curves are accurately recorded. In addition, a new standard, ASTM C 1399, uses similar specimens to determine only the residual strength of FRC.

In ASTM C 1018, "toughness indices" I_5, I_{10}, and I_{20} are calculated as indicated in Figure 22.10a, and one or more of these indices are often specified as acceptance criteria. In addition, the "residual strengths" $R_{5,10}$ and $R_{10,20}$ are usually determined, which represent the average post-cracking load that the specimen may carry over a specific deflection interval. They are presented as a percentage of the load at first crack and are calculated from the toughness indices:

$$R_{5,10} = 20(I_5 - I_{10}) \text{ and } R_{10,20} = 10(I_{20} - I_{10}) \qquad (22.3)$$

In JSCE SF-4, the area under the load-deflection curve out to a specified deflection ($\delta_{tb} = L/150$) is measured and is referred to as the *toughness*. The toughness factor (or equivalent flexural strength), which is a measure of the average residual strength, is calculated as

$$\text{toughness factor} = \text{toughness} \times L/(BH^2 \times \delta_{tb}) \qquad (22.4)$$

where the symbols are as defined in Figure 22.10b.

In ASTM C 1399, the concrete beam is cracked in a standard manner, by loading it in bending in combination with a steel plate (Figure 22.11). The purpose of the steel plate is to prevent complete failure of the beam when it begins to crack. The steel plate is then removed, and the specimen is reloaded to obtain a plot of the reloading load versus the deflection. The average residual strength of the beam over the deflection range of 0.5 to 1.25 mm (0.02 to 0.05 in.) can then be determined.

While ASTM C 1018, JSCE SF-4, and ASTM C 1399 can all be used when writing concrete specifications, care should be taken when doing so. Ideally, any toughness or residual strength parameter should satisfy the following criteria:

1. It should have a physical meaning that is both readily understandable and of fundamental significance if it is to be used for the specification or quality control of FRC.

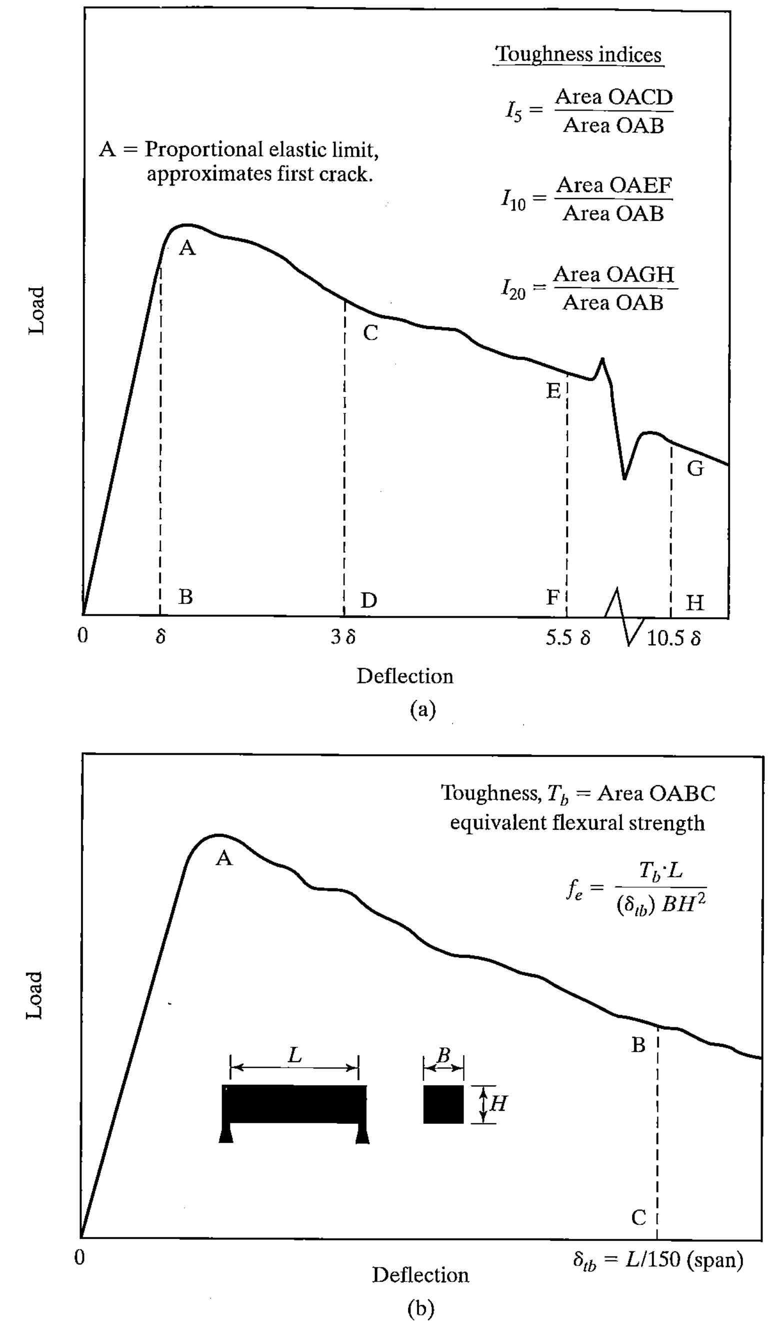


FIGURE 22.10

Schematic of load versus deflection curve and definition of toughness parameters according to (a) ASTM C 1018 and (b) JSCE SF-4.

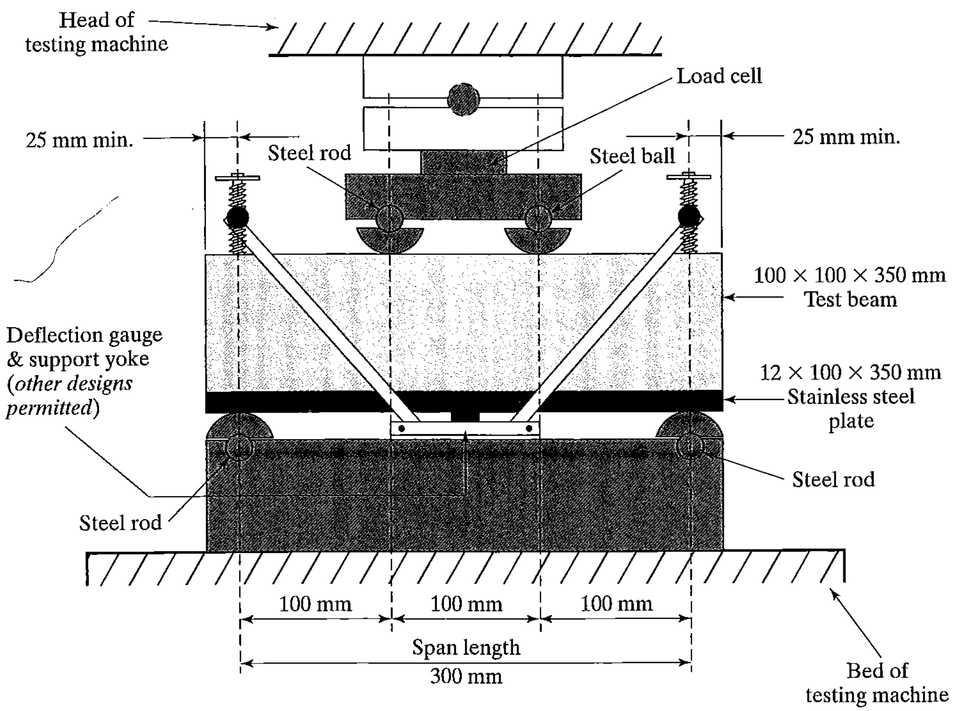


FIGURE 22.11

Apparatus for determining the residual strength of FRC according to ASTM C 1399. Copyright ASTM INTERNATIONAL. Reprinted with permission.

2. The "end-point" used in the calculation of the toughness parameters should reflect the most severe serviceability conditions anticipated in the particular application.
3. The variability inherent in any measurement of concrete properties should be low enough to give acceptable levels of both within-batch and between-laboratory precision.
4. It should be able to quantify at least one important aspect of FRC behavior (e.g., strength, toughness, or crack resistance) and should reflect some characteristics of the load-deflection curve itself.
5. It should be as independent as possible of the specimen size and geometry.

Unfortunately, neither the ASTM nor the JSCE toughness tests can fulfill all of these criteria, and it is important to understand the difficulties encountered when using these methods.

Limitations of ASTM C 1018.

1. It is essential that the load-deflection curve be determined precisely, given the nature of the calculations of the toughness indices. Since the deflections out to the point of first crack (and generally out to the peak load) are very small, it is essential to correct for the "extraneous" deflections due to seating of the specimen on its supports and machine deformations. If this is not done (and in practice this is all too often the case), then spurious values will be determined for the toughness indices. Extraneous deformations can be accounted for in several ways, either by using a special loading frame (the so-called "Japanese yoke" as specified in JSCE SF4), or by mounting dial gauges not only at the midpoint of the specimen, but also over the specimen supports, so that the support deflections can be subtracted from the mid-point deflection. As a quick check that this is being done properly, the measured first-crack deflection for third-point loading of the specimen should be compared with that calculated assuming elastic behavior up to first crack. The deflection is given by

$$\delta = \frac{23\,PL^3}{1296\,EI}\left[1 + \frac{216D^2(1+\nu)}{115L^2}\right] \tag{22.5}$$

where P is the first crack load, L is the span, E is the elastic modulus of the FRC, I is the moment of inertia, D is the specimen depth, and ν is Poisson's ratio. If the measured and calculated values are different by more than a few percent, then it is likely that there is something wrong with the deflection measurements.
2. The toughness parameters are dependent upon the way in which the point of "first crack" is determined; it is often difficult to define this point accurately and unambiguously (particularly for high-toughness FRC).
3. An *instability* usually occurs in the measured load-deflection curves for low-toughness FRC's, unless either a closed-loop servo-controlled testing system or an extremely stiff testing system is used. In this case, the toughness indices I_5, I_{10}, and I_{20}, and their associated residual strength factors, may not be meaningful, and different loading systems may lead to significantly different calculated values.
4. The toughness parameters are not independent of the specimen dimensions.
5. The ASTM toughness parameters are sometimes not particularly sensitive in distinguishing between different fiber contents or fiber types, due to the variability inherent in trying to determine the point of first crack and to the difficulties induced by the instability just mentioned.

Limitations of JSCE SF-4.

1. The toughness parameters are dependent on the specimen dimensions.
2. Since this test provides a "smeared" approach to calculating the toughness parameters, the parameters cannot distinguish between precrack and postcrack behavior;

FRC mixes with quite different load-deflection curves may display similar JSCE parameters.

3. The fixed endpoint ($L/150$) chosen for the test can neither reflect the characteristics of the load-deflection curve, nor be used for different deflection/serviceability conditions.
4. The parameters are age dependent up to an age of about 60 days.
5. On the other hand, the JSCE toughness parameters are little influenced either by different loading systems or by extraneous deformations, and the instability in the load-deflection curves mentioned earlier has only a small effect on the calculated values.
6. As well, the JSCE parameters are often more sensitive than the ASTM parameters in distinguishing among different fiber contents or fiber types.

Limitations of ASTM C 1399.

1. For different types of FRC, the precracks obtained with this loading system may be somewhat different in length, making comparisons of the residual load-bearing capacities between different FRC beams difficult.
2. Simple beam theory cannot properly be used to calculate the strength of a cracked section, so it is not clear what the residual strength determined by this method actually means.

Thus, in any particular case, it may be difficult to interpret the results obtained with either the ASTM or JSCE test methods.

"Template" approach. It has been suggested that a "template" approach be used to specify FRC. Using this approach, *toughness performance levels*, such as those shown in Table 22.3, are defined for flexural tests using the standard 100 × 100 × 350 mm (4 × 4 × 14 in.) test specimens. These are illustrated graphically in Figure 22.12. Toughness performance levels I, II, III, and IV would then represent increasing levels of performance. Then it would only be necessary to compare the template with the actual load-deflection curve to see whether the FRC satisfied the required toughness performance

TABLE 22.3 Toughness Performance Levels for Use with the "Template" Approach [100 × 100 × 350 mm (4 × 4 × 14 in.) test specimen]

Toughness Performance Level	*Residual Flexural Strengths at Deflections (by % of Design Flexural Strength)*	
	1/600 Span (0.5 mm)	*1/150 Span (2.0 mm)*
0	No fiber reinforcement	
I	15%	5%
II	30%	15%
III	50%	30%
IV	75%	45%

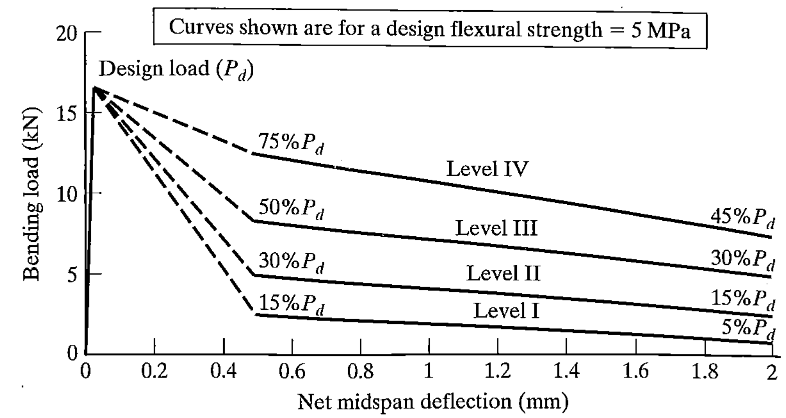


FIGURE 22.12

Template approach to specifying toughness in terms of residual strengths.

level. (Of course, designers could establish their own templates if they so desired.) This method has the advantage that it is not sensitive to the location of the first crack, to the occurrence of extraneous deflections, or to the instability in the measured load-deflection curves, since the exact shape of the curve out to a deflection of 0.5 mm (0.02 in.) is not considered.

At this time, there continues to be considerable controversy as to how the toughness of FRC should be defined or measured. The reader would be well advised to consult the most recent test standards and FRC specifications before embarking on these types of measurements.

Impact Resistance

The impact resistance of plain concrete, which is quite low, can be increased dramatically (by more than an order of magnitude) by the addition of fibers. Steel and carbon fibers are more effective in this regard than synthetic fibers, but all types of fibers increase both the fracture energy and generally the peak loads under impact. Though there is as yet no "standard" impact test for FRC, similar results are obtained with the various tests that have been used: Charpy (pendulum) tests, drop-weight tests, and explosive loading tests. At the higher loading rates associated with impact tests, in which failure occurs in a few milliseconds rather than in a few minutes as in static tests, a higher number of fibers appear to break rather than to pull out. This is related to the bond characteristics of the fibers; a great deal of research is currently underway to try to optimize fiber geometry for the case of impact loading.

Related to impact resistance, it is generally found that fiber additions also improve both the *abrasion* resistance and the resistance of concrete to *cavitation* damage.

Fatigue

The flexural fatigue strength of plain concrete (out to 2 million cycles of loading) is about 55% of the static strength, for all types of loading. A properly designed FRC, however, will display fatigue strengths of from 65% to 90% of the static load. This improvement is

due largely to the effectiveness of the fibers in "tying" the cracks together, and thus inhibiting crack extension during the loading cycles.

Creep and Shrinkage

Up to fiber volumes of about 1%, fiber additions have little or no effect on either the creep characteristics or the drying shrinkage of concrete, though they tend to reduce the crack widths significantly during shrinkage. However, fibers can be very effective in reducing the *plastic shrinkage* (see Chapter 16). Indeed, this is one of the principal uses of polypropylene fibers in modern FRC technology.

Durability

Durability is as important as strength in determining the suitability of concrete for any specific application. Generally, durable concrete should be dense and impermeable. It has been found that well-made FRCs are slightly less permeable than plain concrete, though the decrease is not large enough to have a significant effect on the overall durability of the concrete.

Synthetic fibers are not subject to corrosion. Steel fibers may, of course, rust. However, it has been found that for good quality FRC, this rusting is confined to the "skin" of the concrete; fibers in the interior of the concrete are protected by the alkaline environment. Thus, even with steel fibers, corrosion leads only to cosmetic effects. If this is a problem in a particular application, stainless steel fibers may be used (though at a considerable increase in cost).

As mentioned earlier in this chapter, ordinary E-glass will be attacked by the alkaline environment, and even FRC made with alkali-resistant glass tends to show some degradation in properties over time. Natural organic fibers are likely to suffer from alkaline, bacterial, or fungus degradation and require special treatment if they are to be used widely in FRC.

22.6 APPLICATIONS OF FRC

Although fibers can improve many of the properties of concrete, they are not a panacea for poor concrete practice. Poorly made FRC is no better than poorly made plain concrete—it is just much more expensive. Indeed, before fibers are recommended for use, serious consideration should be given to whether the concrete properties in question can be improved sufficiently by changes in mix design (e.g., by reducing the *w/c* ratio) or in construction procedures (e.g., by improving the curing of the concrete). Nonetheless, where the additional costs can be justified, fibers do have an important role to play in modern concrete technology. Steel FRC has been used extensively for pavements and for highway and airport runway overlays, to reduce both thickness and cracking. Currently, it is used most commonly in industrial floors to reduce damage due to abrasion and impact. It is also becoming increasingly common for tunnel linings (where it is applied as fiber shotcrete) and in repair applications. Polypropylene FRC is used largely to reduce cracking due to plastic shrinkage, to improve the watertightness of water-retaining structures, and to improve the toughness of the concrete. Other fiber types are used in a variety of specialized applications.

Structural Applications of FRC

Fibers cannot be used simply to *replace* conventional reinforcement in concrete. However, there are many applications in which fibers might be used *in conjunction with* steel reinforcing bars to bring about significant improvements in the behavior of reinforced concrete structures or structural elements. Fibers act effectively with steel reinforcement, not only through their role in bridging cracks, but also through their ability to increase the bond between the steel bars and the concrete. Thus, for instance, it has been found that fibers can be used to improve the resistance of reinforced concrete structures to seismic (earthquake) forces. They can also improve the shear behavior of beams and columns.

Unfortunately, at the time of writing (2002), fibers are very little used in truly structural applications. This is in large part because most structural design codes, such as ACI 318, are based on concrete *strength* as the principal design criterion. The current codes are simply unable to deal with *toughness*. That is, they are concerned primarily with the peak loads that a structure might withstand, rather than the "post-peak" behavior. Since it is in the post-peak regime that fibers are most effective, it would require significant changes in the codes for fibers to be used in conjunction with steel in structural design. This is an area that requires much more research and development.

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PROBLEMS

22.1 Why do we add fibers to concrete?
22.2 How do fibers affect the strength of concrete in (a) tension, (b) compression, (c) flexure, and (d) impact loading?
22.3 How do fibers affect crack propagation in concrete?
22.4 By what mechanisms do fibers increase the toughness of concrete?
22.5 What is the significance of the aspect ratio of fibers?
22.6 What are the difficulties associated with the production of fiber reinforced concrete?
22.7 What problems are associated with the use of glass fibers in concrete?
22.8 Compare and contrast the role of high modulus of elasticity fibers (steel or carbon) with that of low modulus (synthetic) fibers in fiber reinforced concrete.
22.9 What effect would you expect fibers to have on (a) plastic shrinkage and (b) drying shrinkage of concrete?
22.10 Under what circumstances might you wish to use fibers in conjunction with conventional steel reinforcement?
22.11 How would you go about trying to optimize a fiber for a particular application?
22.12 If a fiber producer sent you a sample of what was claimed to be a "new and improved" fiber, how would you evaluate this new fiber?

APPENDIX

Tests and Specifications for Concrete and its Constituents

ASTM stands for American Society for Testing and Materials.
CSA stands for Canadian Standards Association.

This is not an exhaustive list. All specifications refer to the latest editions.

Designation	ASTM	CSA
Cements and Mortars		
Specifications		
Blended cements	C 595	A362
Blended cement, performance	C 1157	
Cementitious slag		A363
Expansive cement	C 845	
Portland cement	C 150	A5
Masonry cement	C 91	A8
Mortar cement	C 1329	
Tests		
Air content of mortar	C 185	
Autoclave expansion of cement (soundness test)	C 151	A5
Bleeding of cement pastes and mortars	C 243	
Cement strength uniformity	C 917	
Chemical analysis of cement	C 114	A5

Designation	ASTM	CSA
Compressive strength of mortars (cube test)	C 109	A5
Compressive strength of mortars (prism test)	C 349	
Density of cement by air permeability	C 188	
Drying shrinkage of mortar	C 596	
Early stiffening of hydraulic cement (paste method)	C 451	
Expansion of mortar bars stored in water	C 1038	
Fineness of cement by the 45-μm (No. 325) sieve	C 430	A5
Fineness of cement (air permeability)(Blaine)	C 204	
Fineness of cement (Wagner turbidimeter)	C 115	
Flexural strength of mortars	C 348	
Flow of mortar	C 1437	A5
Flow table	C 230	
Heat of hydration of cement	C 186	A5
Length change of mortars exposed to sulfates	C 1012	
Normal consistency of cement	C 187	A5
Mechanical mixing of cement pastes and mortars	C 305	A5
Optimum SO_3 in portland cement	C 563	
Phase determination of clinker using x-ray powder diffraction analysis	C 1365	
Phase determination of clinker using microscopical point-count procedure	C 1356	
Potential sulfate expansion of mortars	C 452	A5
Restrained expansion of expansive cement mortar	C 806	
Sampling of cement	C 183	A5
Time of setting by Gillmore needles	C 266	
Time of setting by Vicat needle	C 191	A5
Time of setting of mortar by Vicat needle	C 807	

Aggregates

Specifications

Designation	ASTM	CSA
Aggregate for radiation-shielding concrete	C 637	
Concrete aggregate	C 33	A23.1
Lightweight aggregates for structural concrete	C 330	

Tests (sampling aggregates)

Designation	ASTM	CSA
Tests (sampling aggregates)	D 75	A23.2–1A
Abrasion resistance (Micro-Deval apparatus)		A23.2–23A A23.2–29A

Designation	ASTM	CSA
Abrasion resistance (Los Angeles machine)	C 131 C 535	A23.2–16A A23.2–17A
Clay lumps and friable particles	C 142	A23.2–3A
Degradation of fine aggregate due to attrition	C 1137	
Flat and elongated particles		A23.2–8A
Evaluation of frost resistance of coarse aggregate in air-entrained concrete	C 682	
Iron staining materials in lightweight aggregates	C 641	
Lightweight particles	C 123	A23.2–4A
Materials finer than 75-μm (No. 200) sieve	C 117	A23.2–5A
Organic impurities, effect on strength	C 87	A23.2–8A
Organic impurities in fine aggregates	C 40	A23.2–7A
Petrographic examination of aggregates for concrete	C 295	
Potential alkali reactivity of aggregates (mortar-bar method)	C 1260	A23.2–25A
Potential alkali reactivity of cement–aggregate combinations (mortar-bar method/prism)	C 227	A23.2–14A
Potential alkali reactivity of carbonate rocks	C 586	A23.2–26A
Potential alkali–silica reactivity of aggregates (chemical method)	C 289	
Potential volume change of cement–aggregate combinations	C 342	
Reducing samples of aggregate to testing size	C 702	
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Soundness of aggregates	C 88	A23.2–9A
Specific gravity and absorption of coarse aggregate	C 127	A23.2–12A
Specific gravity and absorption of fine aggregate	C 128	A23.2–6A
Surface moisture content of fine aggregate	C 70	A23.2–11A
Total moisture content	C 566	
Uncompacted void content of fine aggregate	C 1252	
Unit weight and voids in aggregate	C 29	
Sulfate ion content in groundwater	D 516	A23.2–2B
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Chemical Admixtures

Specifications

Designation	ASTM	CSA
Admixtures for shotcrete	C 1141	
Air-entraining admixtures	C 260	

Designation	ASTM	CSA
Chemical admixtures	C 494	
Chemical admixtures for flowing concrete	C 1017	
Tests		
Air-entraining admixtures	C 233	

Mineral Admixtures

Designation	ASTM	CSA
Specifications		
Fly ash and calcined natural pozzolans	C 618	
Ground granulated blast-furnace slag	C 989	
Silica fume	C 1240	
Supplementary cementing materials		A23.5
Tests		
Fly ash and natural pozzolans	C 311	
Hydraulic activity of ground slag by reaction with alkali	C 1073	
Mineral admixtures or ground blast-furnace slag effectiveness in preventing excessive expansion due to alkali–silica reaction	C 441	

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Designation	ASTM	CSA
Tests	C 94	

Fresh Concrete

Designation	ASTM	CSA
Specifications		
Concrete made by volumetric batching and continuous mixing	C 685	
Fiber reinforced concrete and shotcrete	C 1116	
Ready-mixed concrete	C 94	
Tests		
Air content (pressure method)	C 231	A23.2–4C
Air content (volumetric method)	C 173	A23.2–7C
Bleeding of concrete	C 232	
Consistency and density of roller-compacted concrete using a vibrating table	C 1170	

Designation	ASTM	CSA
Density of unhardened and hardened concrete by nuclear methods	C 1040	
Flow of freshly mixed concrete	C 1362	
Flow of fiber-reinforced concrete	C 995	
Making and curing test specimens in the field	C 31	A23.2–3C
Making and curing test specimens in the laboratory	C 192	A23.2–2C A23.2–3C
Making and curing test specimens of no-slump concrete		A23.2–12C
Sampling	C 172	A23.2–1C
Slump	C 143	A23.2–5C
Temperature	C 1064	
Time of setting by penetration resistance	C 403 C 1117	
Unit weight, yield, and air content (gravimetric)	C 138	A23.2–6C

Hardened Concrete

Designation	ASTM	CSA
Tests		
Abrasion resistance	C 418 C 779 C 944 C 1138	
Absorption		A23.2–11C
Bond of epoxy-resin systems by slant shear	C 882	
Bond of latex systems by slant shear	C 1042	
Break-off number	C 1150	
Capping cylindrical concrete specimens	C 617	
Cement content	C 1084	
Chloride content (acid-soluble)	C 1152	
Chloride content (water-soluble)	C 1218	A23.2–4B
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Compressive strength, accelerated testing	C 684	A23.2–10C
Creep in compression	C 512	
Critical dilation due to freezing	C 671	
Density, absorption, and voids	C 642	
Density of unhardened and hardened concrete by nuclear methods	C 1040	
Drilled cores and sawed beams	C 42	A23.2–14C
Electrical indication of concrete's ability to resist chloride ion penetration	C 1202	

Designation	ASTM	CSA
Estimating concrete strength by maturity method	C 1074	
Evaluation of frost resistance of coarse aggregates in air-entrained concrete	C 682	
Examination and sampling of hardened concrete in constructions	C 823	
Flexural strength (center-point loading)	C 293	
Flexural strength (third-point loading)	C 78	A23.2–8C
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Fundamental transverse, longitudinal, and torsional resonant frequencies	C 215	
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Length change due to alkali–carbonate reaction	C 1105	
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Pulse velocity	C 597	
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Resistance to rapid freezing and thawing	C 666	
Restrained expansion of shrinkage-compensating concrete	C 878	
Scaling resistance of surfaces exposed to deicing chemicals	C 672	
Splitting tensile strength	C 496	A23.2–13C
Static modulus of elasticity and Poisson's ratio	C 469	
Unbonded caps for cylindrical concrete specimens	C 1231	
Unit weight of lightweight concrete	C 569	
Volume change potential of cement–aggregate combinations	C 342	
Water-soluble chloride content	C 1218	

Solutions to Numerical Problems

CHAPTER 3

3.1 (a) $C_3S = 46.6$, $C_2S = 27.5$, $C_3A = 8.0$, $C_4AF = 11.0$;
(b) $C_3S = 50.4$, $C_2S = 23.2$, $C_3A = 8.0$, $C_4AF = 11.0$;
(c) $C_3S = 42.6$, $C_2S = 30.6$, $C_3A = 11.5$, $C_4AF = 8.0$;
(d) $C_3S = 42.3$, $C_2S = 34.3$, $C_3A = 0$, $C_4AF = 18.4$ (case B)

3.2 (a) 258.1 J/g; (b) 189.2 J/g

3.3 (a) Type III; (b) Type I; (c) Type V; (d) Type IV

CHAPTER 4

4.9 (a) $V_{hp} = 0.54$ cm^3/g; (b) $P_c = 0.21$ cm^3/g; (c) $X = 0.77$

4.10 (a) $V_{hp} = 0.40$ cm^3/g; (b) $P_c = 0.28$ cm^3/g; (c) $X = 0.62$

4.11 (a) $V_{hp} = 0.51$ cm^3/g; (b) $P_c = 0$ cm^3/g; (c) $X = 1$

4.12 (a) $V_{hp} = 0.54$ cm^3/g for each case;
(b) $P_c = 0.02, 0.16, 0.26, 0.34$ cm^3/g;
(c) $X = 0.98, 0.83, 0.72, 0.64$

CHAPTER 7

7.3 (a) 1.96; (b) 3.52

7.4 (a) 2.91; (b) 2.73

7.5 2.72

7.6 $W_{SSD} = 1007$ g; EA = 0.7%

7.7 2.59

7.8 $W_{SSD} = 490$ g; MC = 1.9%

7.9 1.52%

7.10 TM = 0.81%; EA = 0.71%

7.11 1.01%

7.12 TM = 2.88%; SM = 1.87%

7.13 8 kg less

7.19 27.8%

CHAPTER 8

8.12 Assume that the largest capillary pore in well-cured concrete has a diameter of 1 μm. Then the pressure required for water to enter the capillary is given by

$$P = \frac{-2\gamma\cos\theta}{r} = \frac{-2.72\cos 120^\circ}{0.5 \times 10^{-6}} = \frac{+2.72(-0.5) \times 10^{-3}\text{mJ/m}^3}{0.5 \times 10^{-6}}$$
$$= 144 \times 10^{-3}\ \text{J/m}^3$$

Pressure of raindrops = $V \times 1.4 \times 10^{-3}$ J/m^3

Therefore, V = 100 km/h (~60 mph)

CHAPTER 10

10.2 w/c = 0.37, σ = 44.7 MPa; 0.42, 40.3 MPa; 0.47, 36.3 MPa; 0.54, 31.4 MPa; 0.61, 27.2 MPa; 0.69, 23.0 MPa; 0.79, 18.7 MPa; 0.41, 5970 lb/in.^{2}; 0.48, 5160 lb/in.^{2}; 0.57, 4280 lb/in.^{2}; 0.68, 3400 lb/in.^{2}; 0.82, 2540 lb/in.^{2}

10.3

Sieve size, mm	q = 0.33	q = 0.5	q = 0.67
0.15	18%	8%	3%
0.3	23%	11%	5%
0.6	29%	15%	8%
1.18	37%	22%	13%
2.36	46%	31%	21%
4.75	58%	44%	33%
9.5	73%	62%	52%
12.5	80%	71%	63%
19.5	92%	88%	85%
25	100%	100%	100%

10.4 100-mm slump, air = 3%; w/c = 0.39. Cement Type I. Cement = 449 kg/m^3, water = 159 kg/m^3, fine aggregate = 692 kg/m^3, coarse aggregate = 1142 kg/m^3

10.5 Cement = 449 kg/m^3, water = 163 kg/m^3, fine aggregate = 679 kg/m^3, coarse aggregate = 1148 kg/m^3.

10.6 Cement = 382 kg/m^3, fly ash = 67 kg/m^3, water = 175 kg/m^3, fine aggregate = 650 kg/m^3, coarse aggregate = 1145 kg/m^3

10.7 4-in. slump, air = 6.0%, w/c = 0.44. Cement Type V (see Table 10.7). Cement = 693 lb/yd^3, water = 256 lb/yd^3, fine aggregate = 1062 lb/yd^3, coarse aggregate = 1831 lb/yd^3

10.8 Cement = 693 lb/yd^3, water = 293 lb/yd^3, fine aggregate = 1091 lb/yd^3, coarse aggregate = 1804 lb/yd^3

10.9 Cement = 624 lb/yd^3, fly ash = 53 lb/yd^3, water = 305 lb/yd^3, fine aggregate = 1031 lb/yd^3, coarse aggregate = 1813 lb/yd^3

10.10 (a) 0.983 m^3; (b) cement 415 kg/m^3, water = 166 kg/m^3, fine aggregate = 664 kg/m^3, coarse aggregate = 1035 kg/m^3, air-entraining agent = 0.31L/m^3; (c) cement = 420 kg/m^3, water = 168 kg/m^3, fine aggregate = 632 kg/m^3, coarse aggregate = 1018 kg/m^3, air-entraining agent = 0.40L/m^3

10.11 (a) 26.46 ft^3; (b) cement = 585 lb/yd^3, water = 281 lb/yd^3, fine aggregate = 1272 lb/yd^3, coarse aggregate = 1691 lb/yd^3, air-entraining agent = 8.2 fl. oz/yd^3; (c) cement = 602 lb/yd^3, water = 289 lb/yd^3, fine aggregate = 1223 lb/yd^3, coarse aggregate = 1657 lb/yd^3, air-entraining agent = 13.7 fl.oz/yd^3.

CHAPTER 13

13.2 (a) Voigt 6.50 × 10^6 lb/in.^{2}, Reuss 4.57 × 10^6 lb/in.^{2}, Hirsch 5.37 × 10^6 lb/in.^{2}, Counto 5.34 × 10^6 lb/in.^{2}, H-S$_{(+)}$ 6.14 × 10^6 lb/in.^{2}, H-S$_{(-)}$ 5.27 × 10^6 lb/in.^{2};

(b) 2 × 10^6 lb/in.^{2} for all;

(c) 0.88 × 10^6 lb/in.^{2}, 0.62 × 10^6 lb/in.^{2}, 0.72 × 10^6 lb/in.^{2}, 0.77 × 10^6 lb/in.^{2}, 0.71 × 10^6 lb/in.^{2}, 0.76 × 10^6 lb/in.^{2}

13.3 (a) Voigt 84.0 GPa, Reuss 55.6 GPa, Hirsch 66.9 GPa, Counto 66.5 GPa, H-S$_{(+)}$ 78.6 GPa, H-S$_{(-)}$ 65.7 GPa;

(b) 20 GPa for all;

(c) 7.2 GPa, 4.8 GPa, 5.7 GPa, 6.2 GPa, 4.6 GPa, 6.1 GPa

13.4 Logarithmic mixture rule: 5.46 × 10^6 lb/in.^{2}, 1.93 × 10^6 lb/in.^{2}, 0.63 × 10^6 lb/in.^{2} Hashin bounds: 6.07 × 10^6 to 4.71 × 10^6 lb/in.^{2}, 1.88 × 10^6 to 2.03 × 10^6 lb/in.^{2} (*Note*: lower bound solution > upper bound solution), 0.77 × 10^6 to 0.62 × 10^6 lb/in.^{2}

13.5 Logarithmic mixture rule: 70.0 GPa, 19.3 GPa, 5.3 GPa
Hashin bounds: 77.6 to 57.3 GPa, 18.8 to 20.2 GPa (*Note*: lower bound solution > upper bound solution), 6.2 to 4.8 GPa

13.6 4.7 × 10^6 lb/in.^{2}

13.7 15.0 GPa

CHAPTER 15

15.3 (a) 31.9 MPa; (b) 33.8 MPa; (c) 37.7 MPa
15.5 46.6 MPa

CHAPTER 16

16.4 653 $\mu\varepsilon$, 0.48 $\times$ 10^{-6}/lb/in.^{2} (70 $\times$ 10^{-6}/MPa)
16.5 (a) 320 $\mu\varepsilon$; (b) 424 $\mu\varepsilon$
16.6 11.998 ft (143.98 in.) $\times$ 7.999 ft (95.99 in.); 0.049 in. and 0.033 in.
16.7 1058 $\mu\varepsilon$ after application of prestress

CHAPTER 19

19.4 96.4 MPa, 13,940 lb/in.^{2}

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